# PROCEEDINGS

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## TECHNICAL PAPERS

AND

## DISCUSSIONS

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* Publication of closing discussion pending.

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## PAPERS



## DISCUSSIONS

# CONTENTS FOR SEPTEMBER, 1940 (*Continued*)



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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

# MAXIMUM PROBABLE FLOODS ON PENNSYLVANIA STREAMS

By Charles F. Ruff,[1] M. Am. Soc. C. E.

Synopsis

A method, with supporting data, for estimating the maximum probable flood from drainage areas of 100 to 6,000 sq miles is presented in this paper. A maximum storm is derived for various seasons and regions of the state from a study of the records of past storms in Eastern United States. The flood hydrographs caused by 100% runoff from this storm on various sizes of a standard watershed are estimated, and means are developed for correcting these hydrographs to give the corresponding flow from an actual watershed.

A comparison of the maximum flood peaks derived from the storm data with the largest recorded past floods shows a general agreement. Although no frequency is assigned to the flood developed, the basis on which it is derived indicates that such floods must be very rare on any one watershed, and are unlikely to be exceeded.

Although made primarily for Pennsylvania streams, several features of the method are generally applicable. The data used cover a large part of Northeastern United States so that, with modifications required by the locality, they should prove useful in other eastern states.

## General

The term "maximum probable flood" is used to describe not the largest flood possible, but a flood so large that the chance of its being exceeded is no greater than the hazards normal to all of man's activities.

In general, the size of floods at a given point on a stream depends on the watershed—its size and shape; its location with respect to sources of storm rainfall; its imperviousness; and, to some extent, on the slope, pattern, and nature of the stream channels that drain it. These watershed factors remain

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **January 15, 1941.**

[1] Engr., Federal Power Comm., Washington, D. C.

the same for every flood on the watershed, but differ for different watersheds. The size of any individual flood, however, depends also on the storm rainfall that causes it—its intensity, duration, and the location of the storm center with respect to the watershed. The condition of the ground at the time the storm occurs—whether it is dry, saturated, or snow covered—is also a factor. These storm factors vary with every flood, and their different combinations, which are common to all watersheds, account for the difference in flood size observed on the given stream. The maximum probable flood results from the maximum probable combination of these factors acting on the given watershed. The flood record of a single stream (which in Pennsylvania will be from 20 to 50 years) is too short to insure that such a combination has occurred there. The shortness of a single record, however, may be offset by using records from a large area, since what is rare on a single stream may be a fairly frequent occurrence within a large area. Before such a procedure can be applied, the storm factors, which are common to all watersheds, must be separated from the watershed factors, which are different for each.

The first part of the paper is concerned with estimating how the three types of storms which cause floods in the state vary in size with their location, which determines the location factor, and what would be the maximum probable size of these storms in various parts of the state. Storms are complex phenomena, the quantity of rain varying with both duration and the area covered. They can be simplified, however, by expressing them in terms of the peak flow which they would produce from a "standard watershed" of variable area, but whose other watershed factors (shape, location, etc.) are kept constant. The storm is assumed to fit the watershed perfectly and to produce 100% runoff. This "standard flood" is computed by means of a synthetic unit graph which takes account of the shape adopted for the standard watershed. The resultant flood, when corrected for differences in shape and location between the standard and an actual watershed, and with the application of a suitable runoff coefficient, becomes the maximum probable flood for the actual watershed.

Conversely, actual floods of record can be transformed to equivalent floods from the standard watershed, thus eliminating differences between individual watersheds and permitting comparison of the variable storm factors.

It will be obvious that, in estimating anything as intangible as the maximum probable flood, simplicity and reasonableness are more important than extreme precision. The only certainty in regard to any future storm is that it will not exactly duplicate those of the past nor, for that matter, those developed herein. Thus, certain short cuts and approximate assumptions are used which might be out of place in estimating a past flood, or predicting a flood in process from known rainfall data.

## Classification of Flood-Producing Storms

A brief description of the topography of the state is pertinent. Pennsylvania lies almost entirely in three drainage basins: The Delaware River, the Susquehanna River, and the Upper Ohio River. High mountains (2,000 to 3,000 ft) separate the Ohio and Susquehanna basins. In the north, lower moun-

tains (1,500 to 2,000 ft) separate the Susquehanna and Delaware basins; in the south, the divide between them is not higher than 500 ft. The western and northern quarters of the state are rugged and mountainous in contrast to the relatively level southeastern quarter. The Ohio Basin lies on the western slopes of the Allegheny Plateau and is sheltered from the moisture-bearing winds from the ocean by mountains to the east but is exposed to storms approaching from the level Ohio Valley to the west. The Delaware Basin, on the other hand, and that portion of the Susquehanna Basin below the point where the river cuts through the mountains, lie on their eastern slopes and on the Atlantic Coastal Plain. This latter area is not only nearer the ocean, but is unprotected by mountains from coastal storms and hurricanes. However, it is not accessible to storms from the west until they have first passed over a considerable elevation.

The nature and origin of the storms causing Pennsylvania floods were investigated by examining the paths of the low-pressure areas for several days preceding each of the five largest floods at all the principal gaging stations in the state. Paths were not available for the older floods, but the paths of about thirty storms that were responsible for some 150 floods at various stations were secured. These were plotted by basins, and separated into winter and summer storms.

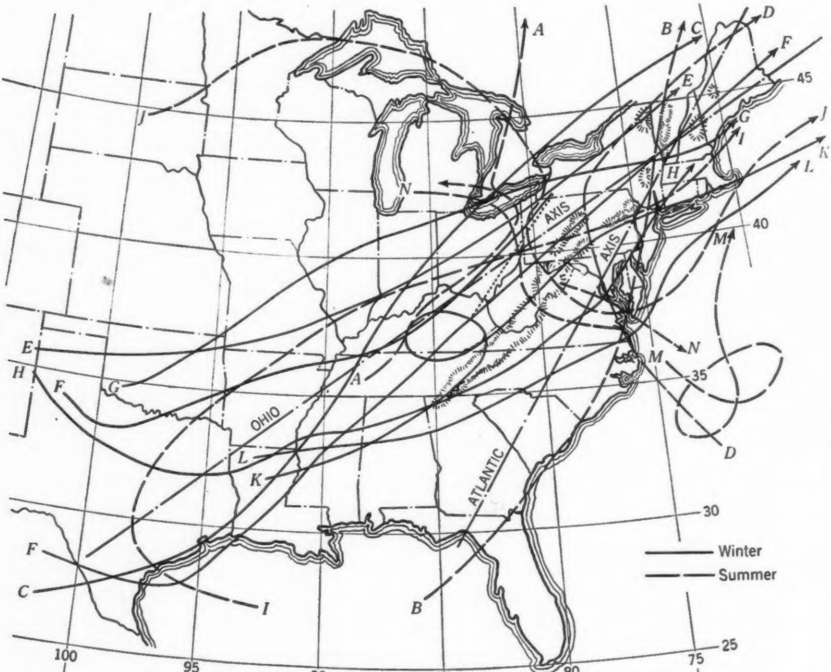
Winter storms were found to follow a well-defined course, moving northeastward up the Ohio Valley. Little difference was observed between the storms affecting the Ohio and the Delaware basins; in many cases, the same storm produced floods in both. Summer storms were more erratic. Those affecting the Delaware Basin and the Lower Susquehanna Basin (the general region east of the mountains) were found to approach from the south or a few points east or west of it. Summer storms affecting the Ohio Basin, Upper Susquehanna, and the headwaters of the Delaware—the region west of the crest of the Appalachian highlands—approached from the west, southwest, or northwest. Table 1 shows a few of the storm paths for some of the larger floods, which are typical. The dates covered and the location of the floods are also given.

On the basis of these data, flood-producing storms were classified into winter storms (occurring from November through April) and summer storms. The latter were further divided into Atlantic Coast storms (those approaching from the south and affecting the southeast corner of the state) and Ohio Valley storms (those approaching from the west and affecting the northern and western parts of the state). Storms *B*, *D*, and *M*, Table 1, are typical of Atlantic summer storms; and storms *A*, *I*, *J*, and *N* are typical of Ohio Valley summer storms. The winter storms are shown by solid lines.

This classification is consistent with the fact that winter storms depend mostly on frontal action, and the precipitation occurs wherever the warm and cold air masses happen to impinge. Summer storms, on the other hand, although subject to frontal action, are often caused by moist, unstable air being forced upward by passing over hills and mountains, and are thus more subject to the influence of the topography.

TABLE 1.—Storms that Caused Great Floods in Pennsylvania

(Curves Show Path of Low Pressure Areas; Winter Storms in Solid Lines, Summer Dotted)




| (a) Winter Storms | | | (b) Summer Storms | | |
|---|---|---|---|---|---|
| Curve | Date | Location of flood | Curve | Date | Location of flood |
| *L* | February 24–27, 1902 | Delaware, Juniata, Lehigh, Schuylkill | *I* | August 17–22, 1888 | Ohio |
| | | | *A* | May 29–June 1, 1889 | Upper Susquehanna, Ohio (Johnstown Flood) |
| *G* | March 13–14, 1907 | Ohio, Juniata | *N* | May 17–20, 1894 | Upper Susquehanna, Schuylkill |
| *F* | March 24–27, 1913 | Ohio, Delaware, Susquehanna | *M* | October 8–11, 1903 | Delaware |
| *C* | Nov. 15–18, 1927 | Delaware, Ohio | *B* | Sept. 29–30, 1924 | Delaware, Schuylkill |
| | | | *D* | August 22–25, 1933 | Lower Susquehanna, Delaware, Schuylkill |
| *E* | Dec. 29–30, 1927 | Ohio | | | |
| *H* | March 15–19, 1936 | Ohio, Susquehanna, Delaware | *J* | July 4–10, 1935 | Ohio, Upper Susquehanna, Upper Delaware, Schuylkill |
| *K* | April 24–29, 1937 | Ohio | | | |

## Estimate of Maximum Probable Daily Rainfall

*Atlantic Summer Storms.*—Storms similar to the Atlantic summer storms causing Pennsylvania floods occur all along the coastal plain, from Maine to Florida. They are larger in the South and smaller in the North. In order to

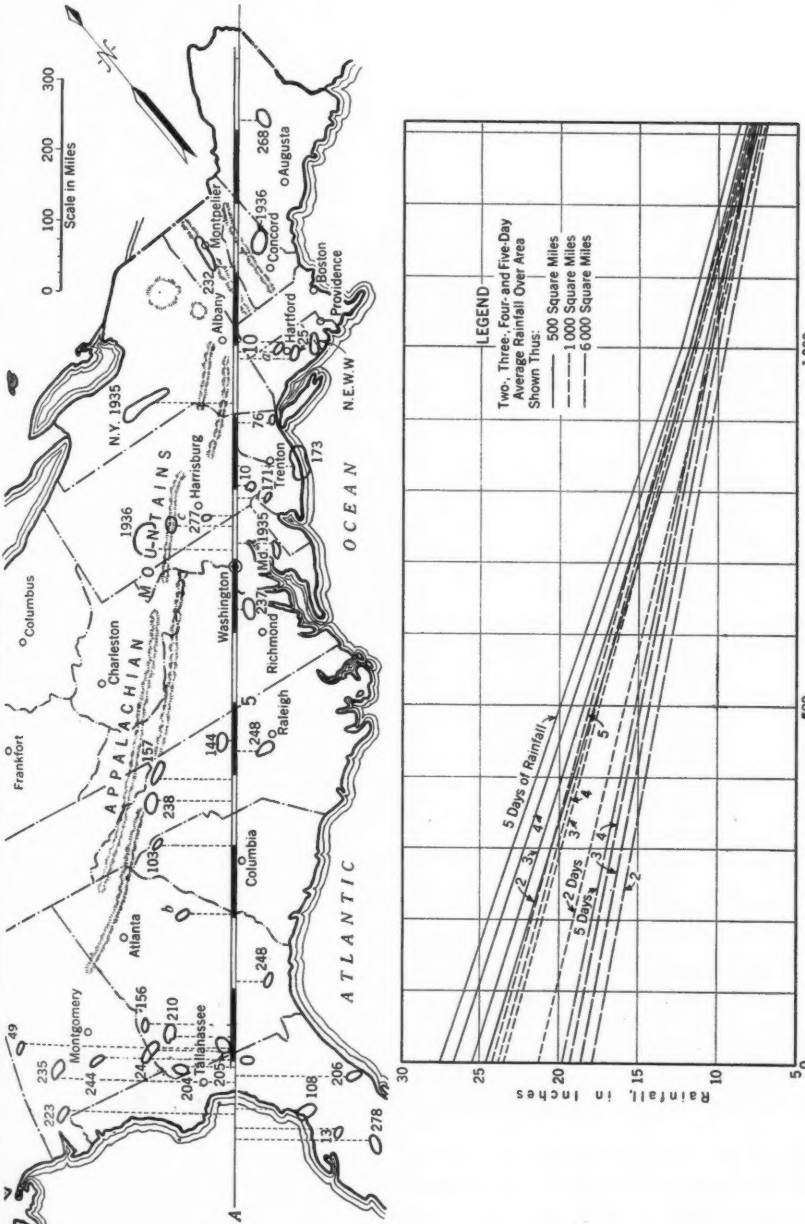



FIG. 1.—STORM RAINFALL ALONG THE ATLANTIC COAST AXIS

TABLE 2.—STORM RAINFALL ALONG THE ATLANTIC COAST AXIS

| Designation on Figs. 1 and 2 | Date | Distance along axis, in miles | Inches of Rainfall: Duration, Two Days, Drainage Areas, in Square Miles: 500 | 1,000 | 6,000 | Duration, Three Days: 500 | 1,000 | 6,000 | Duration, Four Days: 500 | 1,000 | 6,000 | Duration, Five Days: 500 | 1,000 | 6,000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 278 | September 4–7, 1933 | −110 | 12.7 | 12.3 | 10.8 | 16.3 | 15.6 | 13.6 | 16.3 | 15.7 | 13.7 | 16.3 | 15.7 | 13.7 |
| 13 | September 24–26, 1894 | −90 | 11.9 | 11.7 | 10.5 | 12.2 | 11.9 | 11.2 | 12.2 | 11.9 | 11.2 | 12.2 | 11.9 | 11.2 |
| 223 | September 18–22, 1926 | −85 | 16.8 | 15.8 | 11.9 | 16.6 | 15.7 | 12.0 | 16.7 | 15.8 | 12.1 | 16.7 | 15.8 | 12.2 |
| 108 | June 29 to July 3, 1909 | −60 | 14.2 | 13.0 | 9.9 | 14.6 | 13.4 | 10.5 | 15.4 | 14.9 | 12.0 | 16.4 | 16.0 | 14.2 |
| 235 | June 2–6, 1928 | −30 | 12.7 | 12.4 | 11.1 | 13.1 | 12.8 | 11.8 | 14.4 | 14.0 | 12.6 | 14.4 | 14.0 | 13.1 |
| 206 | October 7–11, 1924 | −25 | 18.9 | 16.7 | 8.9 | 19.3 | 17.2 | 9.9 | 19.5 | 17.4 | 10.7 | 21.5 | 19.0 | 11.6 |
| 204 | September 13–17, 1924 | −10 | 12.4 | 11.9 | 10.1 | 12.5 | 12.2 | 10.5 | 13.7 | 13.0 | 10.5 | 13.7 | 13.0 | 10.9 |
| 244 | March 12–15, 1929 | 00 | 23.5 | 21.6 | 15.6 | 25.5 | 23.6 | 17.7 | 25.5 | 23.6 | 18.1 | 25.5 | 23.6 | 18.1 |
| 205 | September 26–30, 1924 | 25 | 10.0 | 9.2 | 7.4 | 10.1 | 9.4 | 8.1 | 10.9 | 10.4 | 8.4 | 11.2 | 10.7 | 8.5 |
| 24 | March 22–23, 1897 | 25 | 11.3 | 11.0 | 9.6 | 11.3 | 11.0 | 9.6 | 11.3 | 11.0 | 9.6 | 11.3 | 11.0 | 9.6 |
| 49 | April 15–17, 1900 | 35 | 11.7 | 11.4 | 10.1 | 17.2 | 11.8 | 11.1 | 17.2 | 11.8 | 11.1 | 17.2 | 11.8 | 11.1 |
| 210 | January 16–19, 1925 | 45 | 8.2 | 8.1 | 7.3 | 10.7 | 10.4 | 9.4 | 11.1 | 10.8 | 9.7 | 11.5 | 11.1 | 9.9 |
| 156 | July 6–10, 1916 | 55 | 15.8 | 14.8 | 11.3 | 18.7 | 17.8 | 13.6 | 20.0 | 19.8 | 17.4 | 20.4 | 19.9 | 17.7 |
| 248 | September 23–27, 1929 | 115 | 16.9 | 16.0 | 12.4 | 18.3 | 17.5 | 12.7 | 18.6 | 17.8 | 13.1 | 18.8 | 18.0 | 13.2 |
| *b* | July 27–31, 1887 | 210 | 12.6 | 11.9 | 8.8 | 14.4 | 13.7 | 10.9 | 14.5 | 13.8 | 11.2 | 15.1 | 14.5 | 12.2 |
| 103 | August 24–26, 1908 | 300 | 12.0 | 11.0 | 8.0 | 13.9 | 13.1 | 11.3 | 13.9 | 13.1 | 11.3 | 13.9 | 13.1 | 11.3 |
| 238 | August 14–18, 1928 | 355 | 10.7 | 10.4 | 8.2 | 11.0 | 10.5 | 8.3 | 11.8 | 11.1 | 8.8 | 11.8 | 11.1 | 8.8 |
| 157 | July 14–16, 1916 | 390 | 18.4 | 17.1 | 12.5 | 18.4 | 17.3 | 12.7 | 18.4 | 17.3 | 12.7 | 18.4 | 17.3 | 12.7 |
| 249 | September 29 to October 3, 1929 | 435 | 10.7 | 10.2 | 9.3 | 10.8 | 10.3 | 9.1 | 12.9 | 12.2 | 9.7 | 12.9 | 12.3 | 9.7 |
| 144 | October 14–15, 1914 | 450 | 8.5 | 7.5 | 5.5 | 8.5 | 7.5 | 5.5 | 8.5 | 7.5 | 5.5 | 8.5 | 7.5 | 5.5 |
| 237 | August 9–13, 1928 | 630 | 11.2 | 10.6 | 8.4 | 11.2 | 10.7 | 8.8 | 11.3 | 10.7 | 8.8 | 12.1 | 11.6 | 9.3 |
| 1936 | March 16–19, 1936 | 710 | .... | .... | .... | 6.3 | 6.0 | 5.0 | 7.0 | 6.9 | 6.0 | 7.0 | 6.9 | 6.0 |
| Md 35 | September 5–6, 1935 | 730 | 13.9 | 13.4 | 11.4 | 15.2 | 14.6 | 12.4 | 15.4 | 14.7 | 12.4 | 15.4 | 14.7 | 12.4 |
| *c* | May 31 to June 1, 1889 | 740 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 |
| 277 | August 20–24, 1933 | 750 | 9.3 | 8.7 | 7.1 | 10.8 | 9.9 | 8.0 | 12.1 | 11.7 | 10.0 | 12.2 | 11.8 | 10.0 |
| 171 | July 19–23, 1919 | 785 | 6.1 | 5.5 | 4.3 | 8.2 | 7.3 | 4.9 | 10.5 | 9.5 | 6.2 | 10.7 | 9.7 | 7.0 |
| 10 | May 18–22, 1894 | 810 | 7.8 | 7.2 | 6.8 | 8.2 | 8.0 | 6.8 | 8.6 | 8.4 | 7.2 | 9.5 | 9.1 | 8.1 |
| 173 | August 13–17, 1919 | 830 | 9.0 | 8.7 | 6.6 | 9.0 | 8.8 | 6.7 | 9.1 | 8.8 | 6.9 | 9.9 | 9.6 | 7.7 |
| 76 | October 8–9, 1903 | 880 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 |
| NY 1935 | July 7–11, 1935 | 915 | 10.0 | 9.8 | 7.5 | .... | .... | .... | 12.9 | 11.9 | 8.4 | .... | .... | .... |
| 25 | July 12–14, 1897 | 975 | 9.5 | 9.1 | 6.9 | 9.6 | 9.3 | 7.0 | 9.6 | 9.3 | 7.0 | 9.6 | 9.3 | 7.0 |
| *a* | October 3–4, 1869 | 980 | 10.4 | 9.7 | 7.8 | 10.4 | 9.7 | 7.8 | 10.4 | 9.7 | 7.8 | 10.4 | 9.7 | 7.8 |
| NEWW | September 16–17, 1932 | 1,005 | 10.4 | 9.0 | 6.7 | 10.4 | 9.0 | 6.7 | 10.4 | 9.0 | 6.7 | 10.4 | 9.0 | 6.7 |
| 232 | November 3–4, 1927 | 1,110 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 |
| 1936 | March 16–19, 1936 | 1,135 | .... | .... | .... | .... | .... | .... | 7.7 | 6.7 | 4.5 | .... | .... | .... |
| 268 | September 16–17, 1932 | 1,315 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 |

display this variation in size, storm profiles were plotted. This method was explained by the writer in 1938. It consists of plotting the rainfall for various conditions of duration and area against the distance along a base line to the point where the storm occurred. The base line used for Atlantic summer storms, designated as the Atlantic Coast Axis, was drawn parallel to the coast and through the center of the area covered by these storms. All the storms available in the "Miami Report" plus a number of more recent storms occurring in and near Pennsylvania were located on a map (see Fig. 1,[1a] and Item (2) in the Appendix), and projected on to the axis. The location is shown by the position of the highest isohyetal line.

Table 2 shows the position and the average precipitation of these storms for the durations and areas used. These storms were all plotted as shown for the 4-day storm on 6,000 sq miles, in Fig. 2. The 1-day rainfall was omitted be-

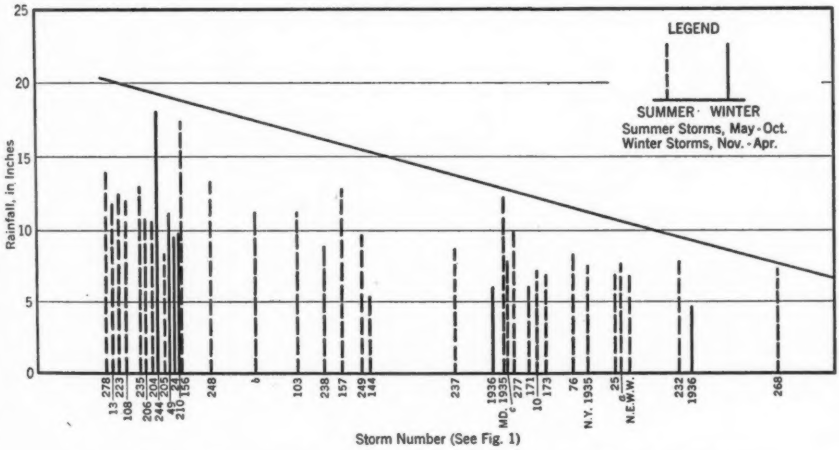



FIG. 2.—ATLANTIC COAST AXIS; 4-DAY STORM RAINFALL ON 6,000-SQ-MILE AREA

cause of the uncertainty as to the actual duration of the rain. The largest storms plotted on these figures showed a marked and fairly uniform decrease in size as they occurred farther to the northeast. Enveloping lines were drawn to pass over the largest storms on the profiles for each condition of area and duration. Straight lines were found to be satisfactory.

The lines for different conditions were adjusted slightly to be consistent with each other—that is, to make the 3-day rainfall more than the 2-day rainfall, and the 500-sq-mile average more than the 1,000-sq-mile average, etc. For this reason, some of the lines pass above all the storms, as in Fig. 2. The enveloping lines for all the conditions of area and duration considered are combined in Fig. 1. Although made for Atlantic summer storms, all available winter storms in the same area were included. Only one winter storm in the far South, No. 244, March, 1929 (storm numbers correspond to those used in the "Miami Report"; see Item (2), Appendix), has any effect on the position of the

[1a] Correction for *Transactions:* At station 4 + 35 in Fig. 1, change designation 248 to 249.

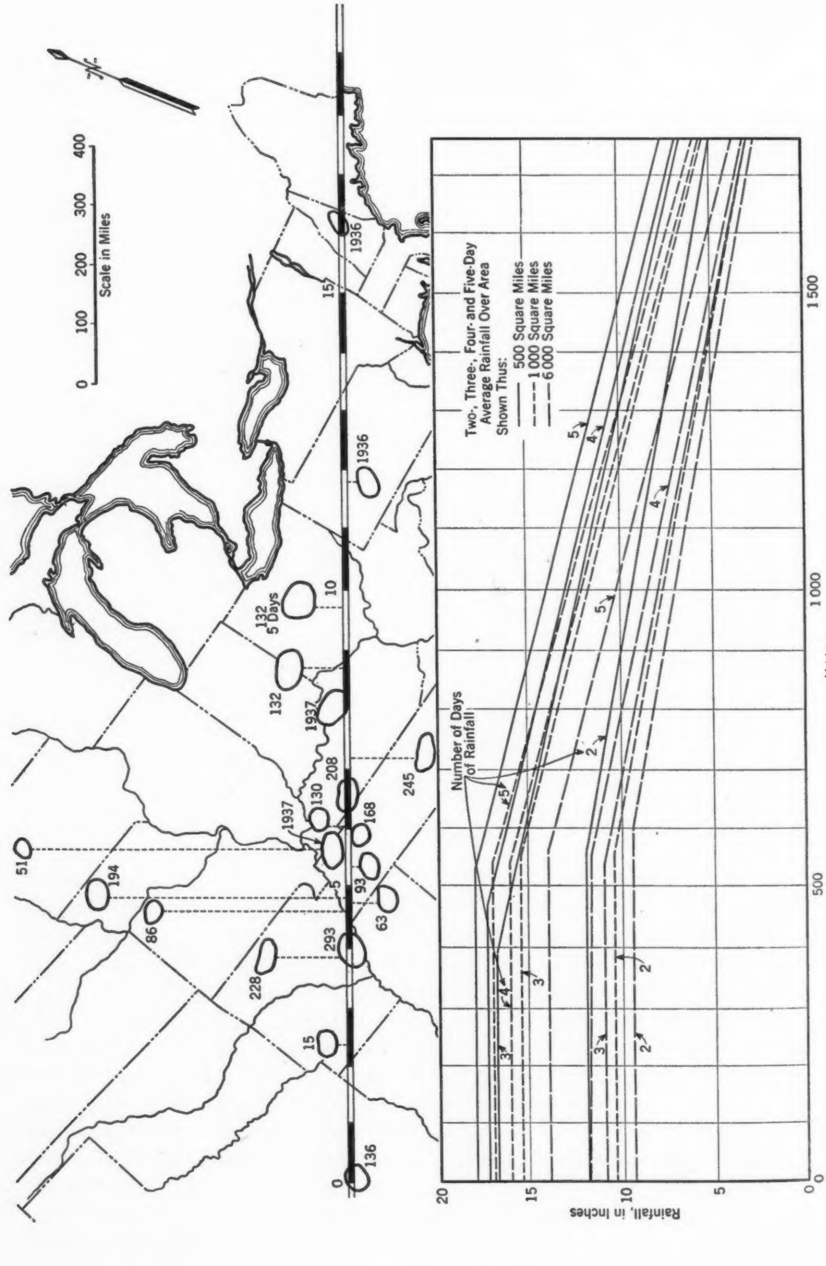



FIG. 3.—STORM RAINFALL ALONG THE OHIO VALLEY AXIS, IN WINTER

enveloping lines. Storm No. 232, the 1927 storm in Vermont, was counted as a summer storm, being distinctly of that type, although it occurred on November 3 and 4.

If the enveloping lines were determined from two storms only, their slope might be ascribed to a chance difference in size between the two storms. However, when several storms at different points support the lines, and others approach them, it suggests a limit to the maximum size of the storms at various points. The trend of decreasing size as the storms occur farther north is thought to be due to the greater distance traveled by the moist air from its source in the tropical Atlantic Ocean or Caribbean Sea. The farther it travels, the greater will be the chance of encountering conditions that will precipitate some or all of its moisture.

The size of these Atlantic storms varies with the season of the year. If the size of a storm is expressed as a percentage of the rainfall indicated by the enveloping line, the effect of its location up or down the coast is eliminated and this seasonal variation is emphasized.

*Ohio Valley Storms.*—For winter storms causing floods in Pennsylvania, a base line called the Ohio Valley Axis was chosen to represent the mean path of the winter storms, as shown in Table 1. All of the winter storms available (see Appendix) which occurred in the region traversed by these paths were considered, and their location plotted in Fig. 3. It is reasonable to suppose that any one of these storms under different conditions might have precipitated its moisture as it passed over the state. The more erratic Ohio summer storms did not show any well-defined average path, and for convenience the same axis was used for them also. The location of the summer storms considered is shown in Fig. 4.[1b] It should be noted that the exact position of the axis selected does not materially affect the result, at least so far as Pennsylvania is concerned. The storm data for both winter and Ohio summer storms are shown in Table 3, and were plotted in the manner illustrated for the 4-day storm on 6,000 sq miles, in Fig. 5. The procedure was the same as in Table 2 and Fig. 2 for Atlantic summer storms. In Table 3 and Fig. 5, however, the summer and winter storms are treated separately.

These storms show the same general trend of decreasing size as the storm occurs farther to the northeast. For the summer storms, however, there is a group near the middle of the axis significantly lower than those at the ends. This is probably due to the distance of this part of the axis from a source of moisture—the Gulf of Mexico or the Atlantic Ocean—and the sheltering effect of the mountains between it and the coast. The summer storms tend to decrease in size as they occur farther from the Gulf of Mexico until a point is reached near enough to the ocean to secure moisture from that source. They then increase as the ocean is approached and finally decrease again as the storms occur farther north along the coast, as shown on the Atlantic Coast profile. This has been interpreted by three series of straight lines enveloping all the storms, although if more data were available it is probable that the breaks in the profile would not be abrupt, but rounded off to some extent. Some of the storms shown for the Atlantic Coast Axis are duplicated in Fig. 5, since its

[1b] Correction for *Transactions:* At station 0 + 70, change designation 157 to 151.

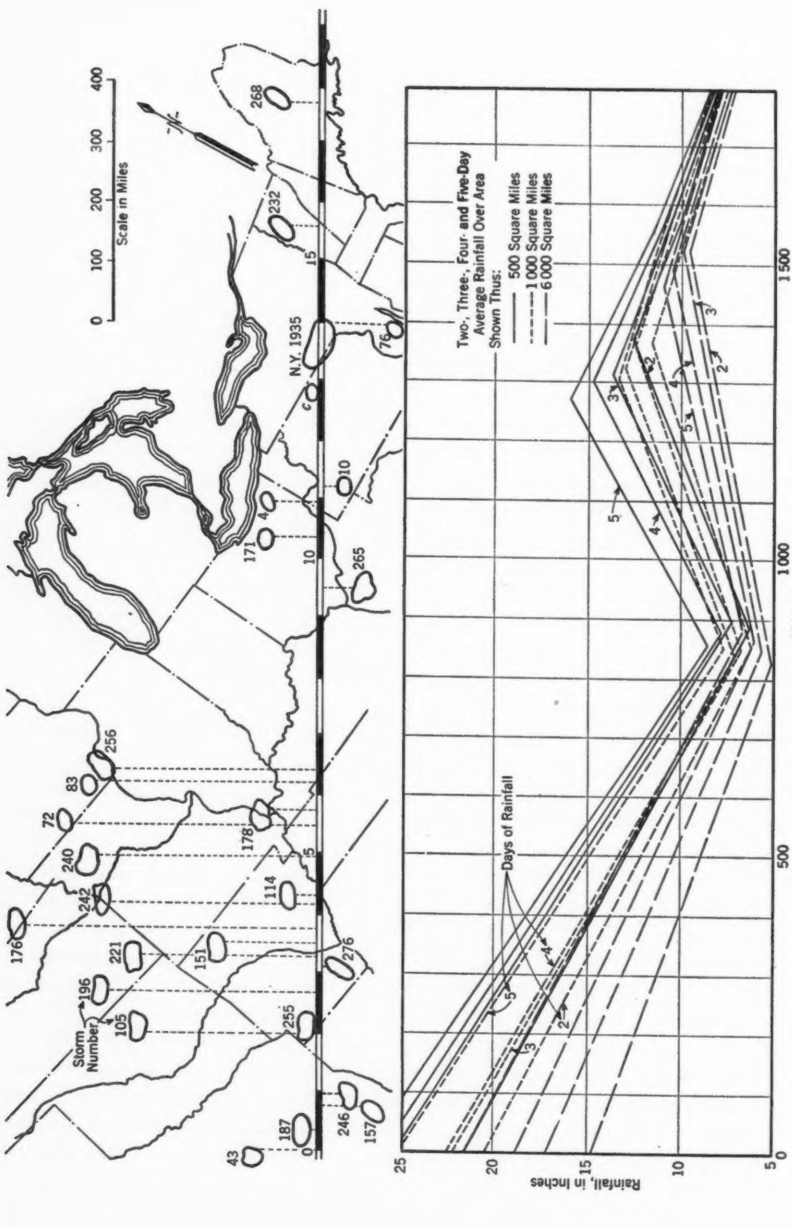



FIG. 4.—STORM RAINFALL ALONG THE OHIO VALLEY AXIS, IN SUMMER

## TABLE 3.—Storm Rainfall Along the Ohio Valley Axis

(*a*) Winter Storms

| Designation on Figs. 3, 4, and 5 | Date | Distance along axis, in miles | Inches of Rainfall: | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Duration, Two Days | | | Duration, Three Days | | | Duration, Four Days | | | Duration, Five Days | | |
| | | | Drainage Areas, in Square Miles: | | | | | | | | | | | |
| | | | 500 | 1,000 | 6,000 | 500 | 1,000 | 6,000 | 500 | 1,000 | 6,000 | 500 | 1,000 | 6,000 |
| 213[a] | Nov. 4-8, 1925 | −50 | 5.4 | 5.0 | ... | 9.5 | 10.2 | 7.9 | 10.7 | 10.3 | 8.9 | 10.9 | 10.3 | 9.1 |
| 136 | Dec. 2-5, 1913 | 0 | 11.2 | 10.5 | 8.4 | 13.3 | 12.8 | 10.9 | 14.4 | 13.8 | 11.5 | 14.4 | 13.8 | 11.5 |
| 15 | Dec. 17-20, 1895 | 235 | 9.3 | 9.0 | 7.7 | 10.7 | 10.5 | 9.2 | 11.0 | 10.5 | 9.3 | 11.0 | 10.5 | 9.3 |
| 228 | April 12-16, 1927 | 380 | 8.9 | 8.5 | 7.4 | 16.3 | 15.0 | 10.4 | 16.4 | 15.0 | 10.6 | 16.5 | 15.3 | 12.2 |
| 253 | Jan. 7-10, 1930 | 400 | 7.8 | 7.5 | 6.8 | 9.8 | 9.6 | 8.5 | 10.1 | 9.9 | 9.1 | 10.1 | 9.9 | 9.1 |
| 242 | Nov. 14-17, 1928 | 420 | 10.9 | 10.6 | 9.3 | 10.9 | 10.6 | 9.4 | 11.1 | 10.8 | 9.5 | 11.1 | 10.8 | 9.5 |
| 63 | Mar. 26-29, 1902 | 470 | 9.0 | 8.9 | 8.4 | 10.3 | 9.7 | 8.7 | 10.6 | 10.1 | 9.2 | 10.6 | 10.1 | 9.2 |
| 93 | Nov. 17-21, 1906 | 530 | 9.9 | 9.5 | 7.9 | 11.7 | 11.1 | 9.8 | 14.8 | 14.0 | 11.4 | 17.8 | 17.0 | 14.1 |
| 1937[b] | Jan. 20-24, 1937 | 555 | ... | ... | ... | 11.2 | 11.0 | 9.8 | ... | ... | ... | 14.4 | 14.1 | 13.0 |
| 168 | Mar. 14-17, 1919 | 580 | 10.0 | 9.5 | 8.3 | 10.6 | 10.3 | 8.6 | 11.5 | 11.0 | 9.0 | 11.5 | 11.0 | 9.0 |
| 130 | Jan. 10-12, 1913 | 600 | 6.8 | 6.6 | 5.8 | 7.5 | 7.4 | 6.9 | 7.5 | 7.4 | 6.9 | 7.5 | 7.4 | 6.9 |
| 208 | Dec. 5-9, 1924 | 660 | 8.0 | 7.9 | 6.8 | 8.0 | 7.9 | 6.9 | 8.0 | 7.9 | 6.9 | 9.1 | 8.8 | 7.6 |
| 245 | Mar. 21-23, 1929 | 720 | 9.5 | 8.9 | 7.0 | 9.5 | 8.9 | 7.0 | 9.5 | 8.9 | 7.0 | 9.5 | 8.9 | 7.0 |
| 1937[b] | Jan. 20-24, 1937 | 800 | 9.1 | 8.9 | 8.1 | 9.8 | 9.7 | 8.9 | ... | ... | ... | 13.0 | 12.9 | 11.7 |
| 132 | Mar. 23-27, 1913 | 870[d] | 8.8 | 8.4 | 7.8 | 9.7 | 9.5 | 8.7 | 10.4 | 10.0 | 9.3 | 10.9 | 10.8 | 9.9 |
| 1936 | Mar. 16-19, 1936 | 1,180 | ... | ... | ... | 6.3 | 6.0 | 5.0 | 7.0 | 6.9 | 6.0 | 7.0 | 6.9 | 6.0 |
| 1936 | Mar. 16-19, 1936 | 1,610 | ... | ... | ... | ... | ... | ... | 7.7 | 6.7 | 4.5 | ... | ... | ... |

(*b*) Summer Storms

| Designation on Figs. 3, 4, and 5 | Date | Distance along axis, in miles | Two Days 500 | Two Days 1,000 | Two Days 6,000 | Three Days 500 | Three Days 1,000 | Three Days 6,000 | Four Days 500 | Four Days 1,000 | Four Days 6,000 | Five Days 500 | Five Days 1,000 | Five Days 6,000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 175[a] | Sept. 14-16, 1919 | −225 | 11.1 | 10.8 | 8.4 | 11.4 | 11.1 | 9.0 | 11.4 | 11.1 | 9.0 | 11.4 | 11.1 | 9.0 |
| 135[a] | Oct. 1-2, 1913 | −80 | 13.7 | 13.3 | 10.2 | 13.7 | 13.3 | 10.2 | 13.7 | 13.3 | 10.2 | 13.7 | 13.3 | 10.2 |
| 43 | June 27 to July 1, 1899 | 0 | 16.6 | 14.6 | 12.3 | 25.6 | 22.1 | 17.5 | 26.1 | 22.4 | 18.4 | 26.1 | 22.4 | 18.4 |
| 187 | Sept. 7-10, 1921 | 40 | 21.1 | 20.0 | 14.2 | 21.1 | 19.8 | 14.2 | 21.8 | 20.5 | 14.4 | 21.8 | 20.5 | 14.4 |
| 151[b] | Aug. 17-20, 1915 | 70 | 16.2 | 15.0 | 11.7 | 19.6 | 19.5 | 17.9 | 19.6 | 19.5 | 17.9 | 19.6 | 19.5 | 17.9 |
| 246 | May 27-31, 1929 | 100 | 10.8 | 10.6 | 8.5 | 11.5 | 11.1 | 8.9 | 11.5 | 11.2 | 9.1 | 11.6 | 11.2 | 9.5 |
| 105 | Oct. 20-24, 1908 | 195 | 11.0 | 10.7 | 8.8 | 12.6 | 12.2 | 10.4 | 14.2 | 13.6 | 11.7 | 14.4 | 13.9 | 11.7 |
| 255 | May 15-19, 1930 | 210 | 7.8 | 7.5 | 5.8 | 11.0 | 10.6 | 8.6 | 12.0 | 11.8 | 10.3 | 12.0 | 11.8 | 10.4 |
| 196 | June 7-11, 1923 | 270 | 7.5 | 6.9 | 5.0 | 7.5 | 6.9 | 5.5 | 7.6 | 7.4 | 6.2 | 7.7 | 7.4 | 6.3 |
| 276 | July 23-27, 1933 | 310 | 7.1 | 6.9 | 6.0 | 11.1 | 10.9 | 9.8 | 16.6 | 16.1 | 14.3 | 19.4 | 18.9 | 16.3 |
| 221 | Sept. 12-15, 1926 | 325 | 8.3 | 7.5 | 5.0 | 8.3 | 7.5 | 5.3 | 8.3 | 7.7 | 5.8 | 8.3 | 7.7 | 5.8 |
| 151[b] | Aug. 17-20, 1915 | 350 | 10.2 | 9.7 | 8.3 | 13.1 | 12.6 | 10.7 | 13.9 | 13.2 | 11.2 | 13.9 | 13.2 | 11.2 |
| 176 | Sept. 17-21, 1919 | 375 | 8.8 | 8.6 | 7.5 | 8.8 | 8.6 | 7.5 | 8.8 | 8.6 | 7.5 | 8.8 | 8.7 | 7.6 |
| 114 | Oct. 4-6, 1910 | 435 | 11.0 | 10.2 | 8.2 | 13.7 | 12.8 | 11.4 | 13.7 | 12.8 | 11.4 | 13.7 | 12.8 | 11.4 |
| 86 | Sept. 15-19, 1905 | 460 | 8.1 | 8.0 | 7.1 | 10.3 | 10.1 | 8.6 | 11.3 | 10.9 | 9.4 | 12.5 | 12.0 | 10.0 |
| 194 | July 8-12, 1922 | 480 | 10.5 | 9.3 | 5.9 | 11.4 | 10.2 | 6.8 | 11.5 | 10.2 | 7.0 | 11.6 | 10.2 | 7.0 |
| 240 | Sept. 10-14, 1928 | 490 | 6.8 | 6.5 | 5.7 | 7.4 | 7.2 | 6.3 | 7.8 | 7.5 | 6.8 | 8.9 | 8.5 | 7.7 |
| 72 | Aug. 25-28, 1903 | 545 | 11.8 | 10.8 | 7.7 | 12.2 | 11.4 | 8.7 | 12.4 | 11.7 | 8.8 | 12.4 | 11.7 | 8.8 |
| 51 | July 14-16, 1900 | 565 | 11.7 | 10.7 | 8.0 | 12.2 | 11.4 | 8.9 | 12.2 | 11.4 | 8.9 | 12.2 | 11.4 | 8.9 |
| 178 | Oct. 26-30, 1919 | 575 | 9.5 | 9.3 | 8.2 | 10.4 | 10.1 | 8.9 | 10.7 | 10.5 | 9.3 | 11.1 | 10.7 | 9.5 |
| 83 | June 9-10, 1905 | 625 | 10.9 | 10.0 | 6.9 | (One Day Storm) | | | | | | | | |
| 256 | June 11-15, 1930 | 640 | 8.5 | 8.0 | 5.8 | 9.3 | 8.6 | 6.2 | 9.3 | 8.6 | 6.3 | 9.3 | 8.6 | 6.3 |
| 265 | July 4-6, 1932 | 950 | ... | ... | ... | 7.5 | 7.0 | 3.0 | ... | ... | ... | ... | ... | ... |
| 171 | July 19-23, 1919 | 1,035 | 6.1 | 5.5 | 4.3 | 8.2 | 7.3 | 4.9 | 10.5 | 9.5 | 6.2 | 10.7 | 9.7 | 7.0 |
| 4 | May 15-17, 1893 | 1,090 | ... | ... | ... | 8.0 | 7.9 | 6.5 | ... | ... | ... | ... | ... | ... |
| 10 | May 18-22, 1894 | 1,120 | 7.8 | 7.2 | 6.8 | 8.2 | 8.0 | 6.8 | 8.6 | 8.4 | 7.2 | 9.5 | 9.1 | 8.1 |
| c | May 31 to June 1, 1889 | 1,275 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 | 9.1 | 8.8 | 7.5 |
| NY 1935 | July 7-11, 1935 | 1,450 | 10.0 | 9.8 | 7.5 | ... | ... | ... | 12.9 | 11.9 | 8.4 | ... | ... | ... |
| 76 | Oct. 8-9, 1903 | 1,495 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 | 11.9 | 10.9 | 8.4 |
| 232 | Nov. 3-4, 1927 | 1,560 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 | 9.1 | 8.9 | 7.7 |
| 268[c] | Sept. 16-17, 1932 | 1,770 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 | 8.0 | 7.9 | 7.2 |

[a] Located in Texas—not shown in Figs. 3 and 4. [b] These storms had two centers. [c] Located in Maine—not shown in Fig. 5. [d] 3-day center; 5-day center moved to 975 miles.

purpose is to show the largest storms in the region covered and not simply the Ohio Valley storms.

The winter storms decrease in size more uniformly and do not show a dip near the center of the axis, where they are larger than the summer storms. Apparently, they vary in size with their distance from the Gulf; the horizontal enveloping lines to the left of Figs. 3 and 5 cover a group of storms roughly equidistant from the Gulf. This part of the profile has little significance for Pennsylvania, and might be revised in a study made for that region. The enveloping lines, as drawn, are consistent with the relative freedom of winter storms from the effect of topography and the absence of large winter storms along the Atlantic Axis. The Ohio Axis profiles are not as clear-cut as the

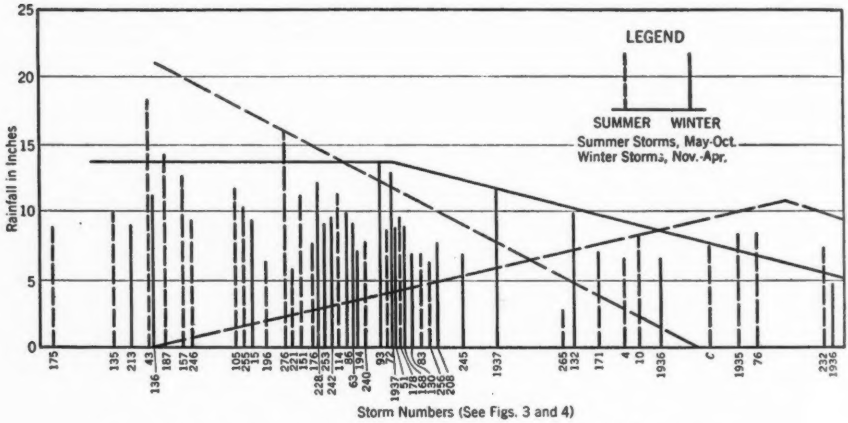



FIG. 5.—OHIO VALLEY AXIS; 4-DAY STORM RAINFALL ON 6,000-SQ-MILE AREA

Atlantic, especially the summer profiles, and certain sets of the plotted data, taken by themselves, appear arbitrary. However, when the entire group of lines is considered together, it is apparent that the lines, while consistent as a group, are in many cases determined by different storms for different conditions of area and duration. No storm, in fact, equals the envelop for every such condition. The combined enveloping lines are shown in Figs. 3 and 4. A plotting of seasonal occurrence of the Ohio Axis storms showed no marked seasonal variation in size aside from the difference between summer and winter storms indicated by the storm profiles.

*Variation of Size with Location.*—The effect of location on different conditions of area and duration seems to be about the same. If, for example, the 4-day, 6,000-sq-mile rainfall at one point is half that at another point, the 2-day, 500-sq-mile rainfall will also be approximately half. If this were exactly so, all the sets of lines on Figs. 1, 3, and 4 would intersect at common points on the base lines. A study of maximum rainfall at single stations over the area covered by the storm profiles led to the conclusion that the small area, or local storms, which might not have been included in the "Miami Report," also varied with location in about the same manner as the large area storms. It

was assumed, therefore, that at least within the state location has the same effect on all conditions of area and duration. The size of the maximum probable rainfall at any point may thus be expressed as a percentage of the maximum probable rainfall at some fixed point of reference, and this percentage may be taken as a constant for all conditions of duration and area. For this purpose, station 12, Ohio Axis, was chosen as the point of reference for storms referred to that axis, and station 8, Atlantic Axis, for Atlantic summer storms.

The variation in size of the maximum probable storm, expressed in percentages of the size of the storm at these reference points, is shown in Fig. 6.

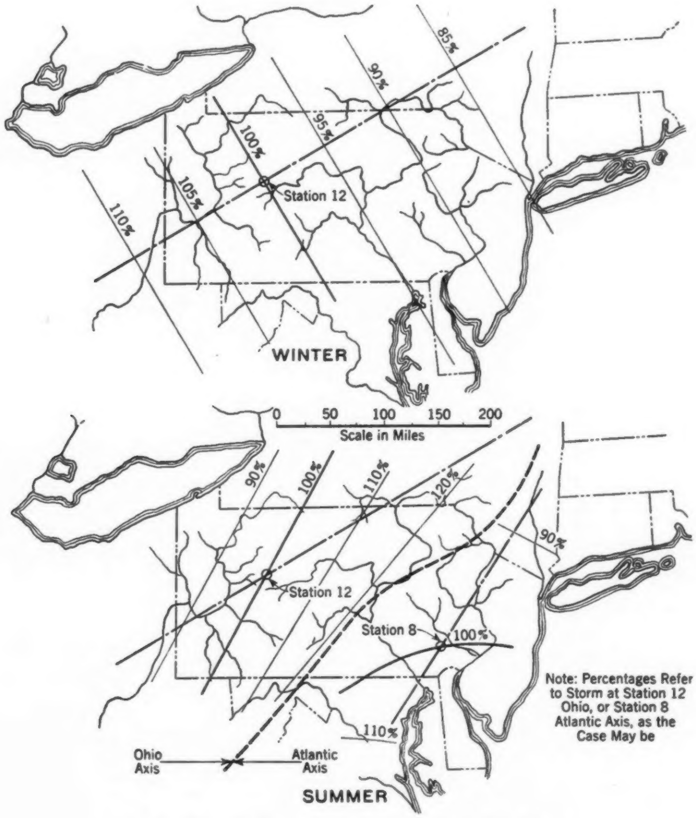



FIG. 6.—VARIATION IN SIZE OF STORM WITH ITS LOCATION

In drawing the winter map, the percentage lines are run at right angles to the axis, thus giving the same values to points having approximately the same distance of travel from the Gulf. For the summer storms the boundary between the Ohio Valley and Atlantic Coast storms was assumed to follow the crests of the mountains, but the latter are not, perhaps, a definite limit to either type of storm. In order to reconcile the values determined along this

boundary from the two different axes, it was necessary to bend the lines away from a right-angle position, giving due regard to the large storms on either side of the axis. The resulting map, of which only the Pennsylvania part is shown, was roughly similar to maps for single-station rainfall shown by the late David L. Yarnell, M. Am. Soc. C. E. (Item (4), Appendix) and the "Miami Report" (Item (3), Appendix).

TABLE 4.—NUMBER OF STORMS TOUCHING THE PROFILE AND THOSE EXCEEDING 90% OF IT DURING 50 YEARS OF RECORD

| Duration of storm, in days | WATERSHED AREAS, IN SQUARE MILES: | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 500 | | 1,000 | | 6,000 | | Total | |
| | Percentage of Envelop Storm: | | | | | | | |
| | 100 | 90 to 100 | 100 | 90 to 100 | 100 | 90 to 100 | 100 | 90 to 100 |
| (*a*) SUMMER STORMS, ATLANTIC COAST AXIS | | | | | | | | |
| 2 | 1 | 7 | 3 | 7 | 1 | 2 | 5 | 16 |
| 3 | 3 | 6 | 3 | 3 | 1 | 3 | 7 | 12 |
| 4 | 0 | 4 | 3 | 3 | 0 | 4 | 3 | 11 |
| 5 | 0 | 4 | 2 | 3 | 0 | 2 | 2 | 9 |
| Total conditions | 4 | 21 | 11 | 16 | 2 | 11 | 17 | 48 |
| Total number of storms involved | .... | .... | .... | .... | .... | .... | 3 | 9 |
| Average number of conditions per storm | .... | .... | .... | .... | .... | .... | 5.67 | 5.33 |
| Average interval between storms, in years | .... | .... | .... | .... | .... | .... | 17 | 5 |
| (*b*) WINTER STORMS, OHIO VALLEY AXIS | | | | | | | | |
| 2 | 1 | 6 | 2 | 5 | 2 | 6 | 5 | 17 |
| 3 | 0 | 1 | 0 | 1 | 2 | 4 | 2 | 6 |
| 4 | 0 | 2 | 1 | 2 | 2 | 6 | 3 | 10 |
| 5 | 1 | 2 | 1 | 2 | 2 | 4 | 4 | 8 |
| Total conditions | 2 | 11 | 4 | 10 | 8 | 20 | 14 | 41 |
| Total number of storms involved | .... | .... | .... | .... | .... | .... | 7 | 13 |
| Average number of conditions per storm | .... | .... | .... | .... | .... | .... | 2 | 3.15 |
| Average interval between storms, in years | .... | .... | .... | .... | .... | .... | 7 | 4 |
| (*c*) SUMMER STORMS, OHIO VALLEY AXIS | | | | | | | | |
| 2 | 4 | 6 | 3 | 6 | 5 | 7 | 12 | 19 |
| 3 | 2 | 3 | 2 | 8 | 3 | 8 | 7 | 19 |
| 4 | 2 | 4 | 2 | 7 | 1 | 5 | 5 | 16 |
| 5 | 0 | 3 | 1 | 2 | 1 | 3 | 2 | 8 |
| Total conditions | 8 | 16 | 9 | 23 | 10 | 23 | 26 | 62 |
| Total number of storms involved | .... | .... | .... | .... | .... | .... | 11 | 15 |
| Average number of conditions per storm | .... | .... | .... | .... | .... | .... | 2.36 | 4.13 |
| Average interval between storms, in years | .... | .... | .... | .... | .... | .... | 5 | 3 |

*Discussion of Storm Frequency.*—The records used in drawing the storm profiles cover, roughly, the period from 1892 to 1937 (see Appendix). Three of the largest storms before 1892 are included, but a number of storms in the early part of the record have probably been missed. For the purpose of discussing the frequency of these storms, it will be sufficiently accurate to assume that the

record covers 50 years. A storm, therefore, whose rainfall equals the amount shown by the enveloping lines at the point where it occurs may be considered to be as large, relative to its position, as any storm of which there is any record during the past 50 years.

Table 4 shows the number of times the enveloping lines have been equaled, and the number of times 90% of the lines have been exceeded for various conditions of duration and area. Of the Atlantic Coast summer storms, three equal the line for seventeen conditions of duration and area, an average of 5.67 conditions for each storm. Since these storms are based on a 50-yr record, the average interval between them may be taken as 17 years. A storm equaling the lines for five or six conditions may thus be expected every 17 years, on the average. Table 4 shows the frequencies with which such storms happen somewhere within the area covered by the records used in drawing the storm profiles. No two of the 100% storms have occurred at the same point, or even opposite the same station on the axis. The irregular distribution of the smaller storms is less significant than the more uniform spacing of the large ones which define the enveloping lines. If it is assumed that a storm equaling the lines on the storm profiles (similar to Figs. 2 and 5) is equally likely to occur at any point along the axis, the chance of a storm occurring in the state can be estimated from the relative size of the areas involved. The total area covered by the Atlantic Summer storms is about 400,000 sq miles. Of this area, 13,000 sq miles lie in the Lower Susquehanna and Delaware basins of Pennsylvania subject to such storms. Thus, there is about one chance in thirty that a storm occurring somewhere in the entire area will occur in the state. If one storm in thirty occurs in the state, the average interval between such storms will be 30 times 17 years or 510 years. The Ohio storms, both summer and winter, covered an area of roughly 780,000 sq miles. On the same basis the approximate frequency of the storms within Pennsylvania is as shown in Table 5.

TABLE 5.—COMPUTED AVERAGE INTERVAL BETWEEN OCCURRENCES OF STORMS IN PENNSYLVANIA

| Drainage Basin | Storm type | Area, in square miles | Season | INTERVAL, IN YEARS | |
|---|---|---|---|---|---|
| | | | | 100% storms | 90% storms |
| Lower Susquehanna and Delaware........ | Atlantic | 13,000 | Summer | 510 | 165 |
| Upper Susquehanna and Ohio............ | Ohio | 32,000 | Summer | 120 | 80 |
| Entire state............................ | Ohio | 45,000 | Winter | 120 | 70 |

These data are not presented as a precise estimate of the frequency of the storms in question. However, they do indicate the rarity of such storms within the state. The chance of such a storm scoring a "bull's-eye" on a particular watershed is obviously even smaller, and the interval between such occurrences on the same watershed would be very much longer than is indicated in Table 5. The foregoing reasoning suggests how data from a wide area may be substituted for the long records (which are not available) to estimate the 100-yr or 1,000-yr flood, provided that the law of variation within the area is understood.

It is realized that the records are incomplete in several respects, and that 50 years is not a long time. It is always possible for past records to be exceeded; but even so, the chance that a storm which equals the storm profile will occur on a particular watershed in the position which causes the greatest flood is very small; and the chance that this storm should also happen to be one of the rarer storms that exceed the profiles for just the particular area and duration which are critical for that watershed must be so small as to be considered improbable. The quantities of rainfall shown in Figs. 1, 3, and 4, therefore, were used as the daily rainfall of the maximum probable storm for the area and season.

## Maximum Probable Rainfall for Short Durations

An estimate of correspondingly large rainfalls for durations shorter than 2 days was then made. The relation between rainfall of the same relative size (or frequency) but of different durations is shown in Fig. 7(*a*),[1c] in which the 100-yr rainfall, in inches, at station 12, Ohio Axis, as taken from the maps in the reports of Mr. Yarnell and the Miami Conservancy District, is plotted against duration. The data were not separated into summer and winter, but the summer storms fix the position of most points. Maximum probable summer rainfall at station 12 for 2 to 5 days, as given by the profiles on Figs. 3 and 4, was then plotted in Fig. 7(*a*). On the assumption that the maximum short-duration rainfall would be in the same ratio to the 100-yr rain as are the longer durations, a line parallel to the 100-yr line was drawn through these points. A number of record rainfalls from the "Morgan Report" (see Item (5), Appendix) were also plotted in Fig. 7(*a*) as a check. The line is seen to pass over, or near, all the points except a 1-hr storm in Cincinnati, Ohio, in 1924.

For the corresponding curve for winter, much less data are available. A search of the hourly records of recent years disclosed nothing greater than 1.5 in. in an hour, but the record is too limited to assume that this approaches the maximum. Therefore, the winter line was drawn simply as an extension of the points from the profiles for durations of 2, 3, 4, and 5 days and with a steeper slope than the summer line, indicating a more nearly uniform rain, which is a characteristic of winter storms.

In Fig. 7(*a*), only the rainfall at a single station was considered. In Fig. 7(*b*), the effect of the size of the area on the average rainfall is introduced. These curves show the maximum rainfall for the period, averaged over the area indicated. Using the rainfall at one station for various periods from 1 hr up, from Fig. 7(*a*), and the 2-day to 5-day rainfall over various areas as taken from the storm profiles, the other values are extrapolated. It is realized that this procedure is approximate; it is resorted to only through lack of better data. For Ohio winter rainfall, the same process was followed in preparing Fig. 7(*c*).

Fig. 8(*a*), for summer rainfall at station 8, Atlantic Axis, corresponds to Fig. 7(*a*) for the Ohio Axis, and was constructed in the same manner. In this case, a number of rainfalls in the Morgan report exceed the assumed maximum line. All these prove to be very old records, the most recent being at

[1c] Correction for *Transactions:* In Fig. 7(*a*), change the subcaption to read "Maximum Rainfall for Various Periods."

Chester, Pa., in 1843. Although these old records are not necessarily in error, the absence of any comparable records in the past 100 years indicates that such storms, if they do occur, are too rare to be termed "probable" for a particular

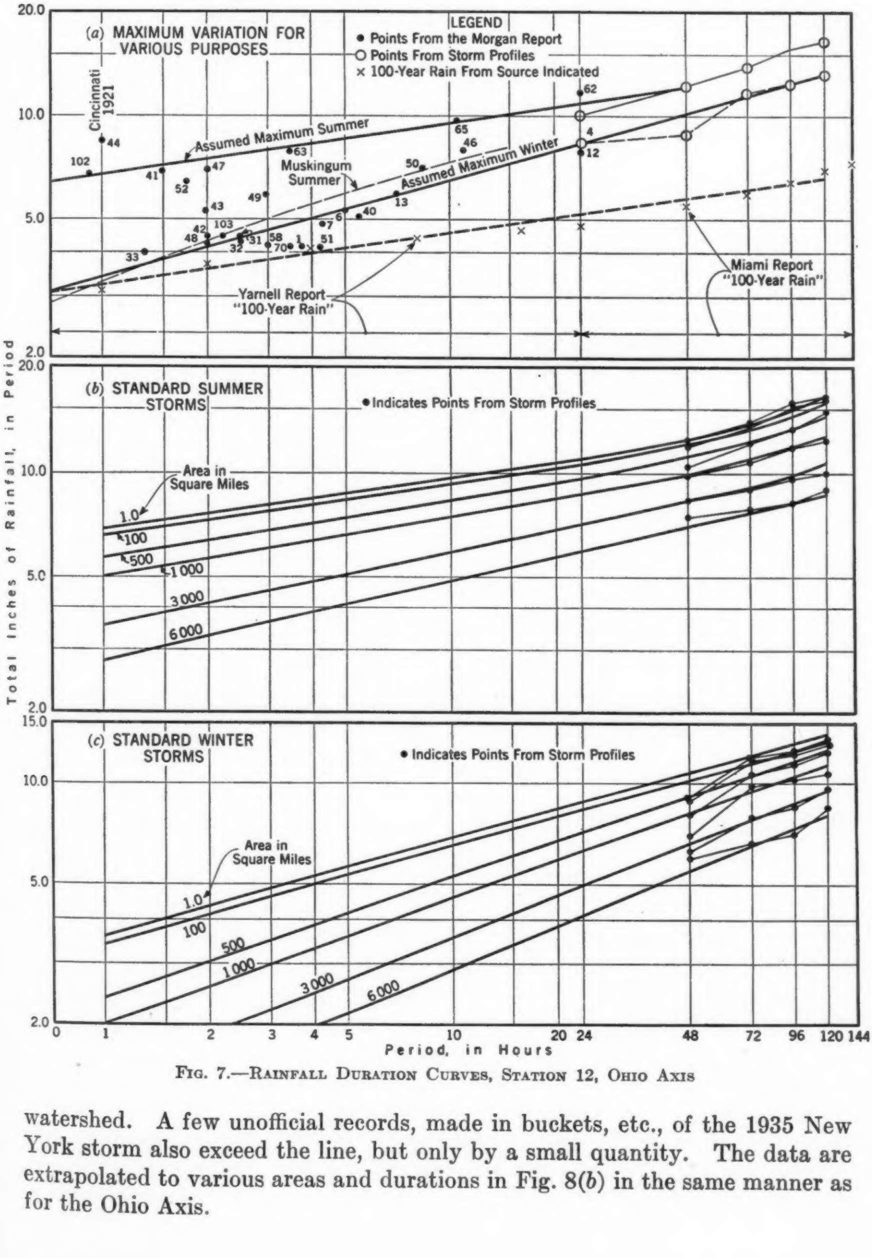



Fig. 7.—Rainfall Duration Curves, Station 12, Ohio Axis

watershed. A few unofficial records, made in buckets, etc., of the 1935 New York storm also exceed the line, but only by a small quantity. The data are extrapolated to various areas and durations in Fig. 8(b) in the same manner as for the Ohio Axis.

The values determined by this extrapolative process which have the greatest error are those for short durations on large areas. However, as the area of the watershed to which the storm is to be applied increases, the importance of the intensity of the rainfall of shorter duration diminishes. There is a gap between the data for short-duration rainfall at a single station, as used in storm-sewer design, and the data on daily rainfall over large areas, as given in the "Miami Report" and elsewhere, which can be filled only when better records become available. The attempt made herein to bridge this gap is far from perfect, but it is thought to be sufficiently accurate for the purpose for which it is used.

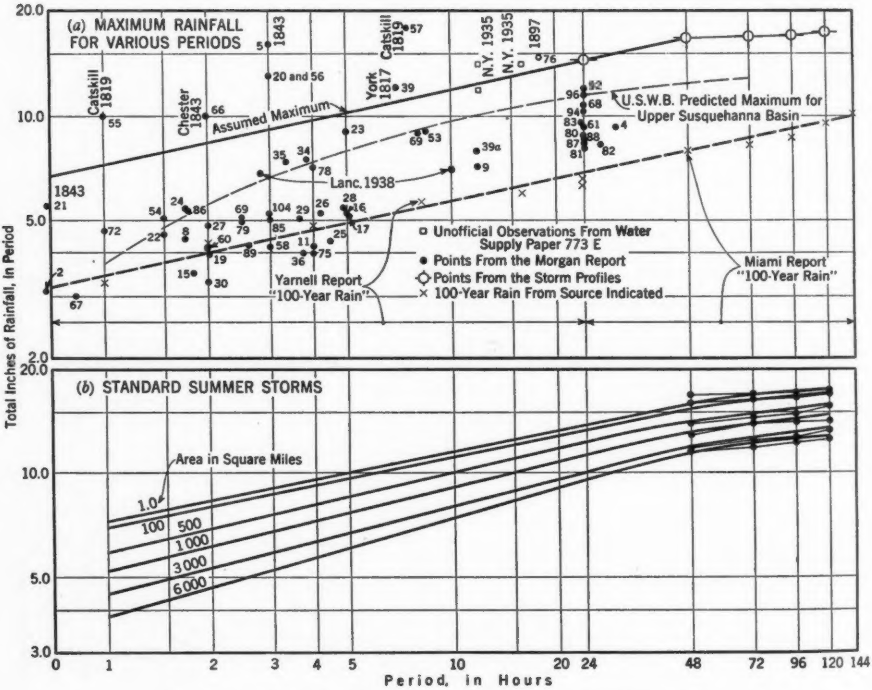



FIG. 8.—RAINFALL DURATION CURVES, STATION 8, ATLANTIC AXIS

The rainfall shown in Figs. 7(*b*), 7(*c*), 8(*b*) was adopted as the maximum probable rainfall at the two reference points.

The conclusions reached from the foregoing storm study may be summarized as follows:

1. Three general storm types—the Atlantic summer and the Ohio summer and winter—are responsible for Pennsylvania floods, and should be treated separately.

2. The occurrence, on a given watershed, of a storm equaling the quantities indicated by the storm profiles (see Figs. 1, 3, and 4) is sufficiently rare to justify their use as a measure of the size of the maximum probable storm.

3. Each type of maximum probable storm varies in size from place to place, as indicated on the profiles, but each may be considered to have the same structure—that is, the same average relation between the different areas and durations, at least throughout the area of the state to which it is applicable.

## Development of the Standard Floods

In reducing the aforementioned rainfall estimate to flood flows on various areas, use is made of three devices—(*a*) the standard storm, (*b*) the standard watershed, and (*c*) the standard flood—which are defined as follows:

(*a*) The standard storm is one that occurs at a particular reference point and equals the maximum probable rainfall at that point for every condition of area and duration. The rain is assumed to occur symmetrically throughout the duration of the storm with the maximum hour in the middle, the maximum 2 days a day before and a day after the middle, etc., as shown in Fig. 9, where the

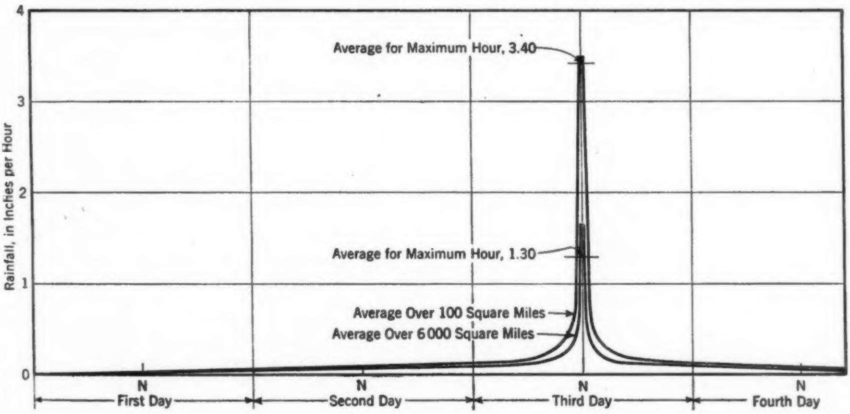



Fig. 9.—Standard Storm (Winter); Station 12, Ohio Axis

rainfall on the fourth and fifth days is symmetrical with the second and first days, respectively. Three standard storms are used: (*a*) Winter and (*b*) Ohio Valley summer, both at station 12, Ohio Axis, and (*c*) Atlantic summer, at station 8, Atlantic Axis.

(*b*) The standard watershed is one that has constant shape and physical characteristics, but variable size. An ellipse, with a ratio of length to average width of four, was used as being an approximation of the average watershed in the state. The length of the stream corresponds to the length of the ellipse (see Fig. 10).

(*c*) The standard flood is produced by 100% runoff from a standard storm occurring on a standard watershed in such a position that the area of greatest rainfall coincides with the watershed. Three types of standard floods are considered, corresponding to the three types of storms. Each type has a series of values for the peak flow corresponding to various areas of the standard water-

shed. Since all standard watersheds are identical, except in size, the standard flood is a convenient means of expressing the size, and other characteristics, of the standard storm in terms of flow from various areas.

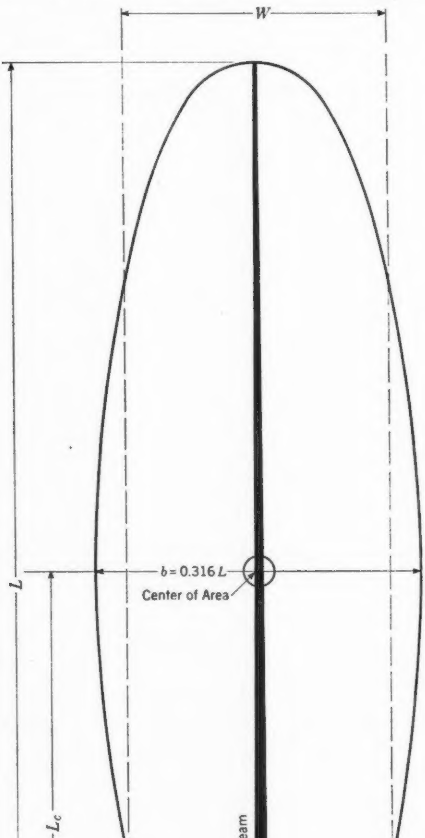



FIG. 10.—DIMENSIONS OF STANDARD WATERSHED

*The Standard Storm.*—The standard storms are developed directly from Figs. 7(*b*), 7(*c*) and 8(*b*). The winter standard storm for both the 100-sq-mile and the 6,000-sq-mile average rainfall is shown in Fig. 9. The other types of standard storms, not illustrated, are very similar.

This distribution in time of the standard storms was not assumed to represent typical or average conditions in actual storms, although it is suggestive of the hourly records of some of the large storms. It was chosen as a distribution which, when applied to a given watershed, would give the largest, or nearly largest, flood. In some cases a different distribution of rainfall might give a slightly larger peak. On the other hand, although an actual storm may equal the standard storm for the area and duration which are most critical for the watershed, past records indicate that it would fall short for other durations. This would tend to offset any increase in the flood peak due to a more adverse distribution of rainfall.

*The Standard Watershed.*—The standard watershed used is shown in Fig. 10. In order to determine the hydrographs resulting from the standard storm, unit hydrographs were developed for various sizes of the shed, following the empirical methods and formulas developed by F. F. Snyder, Jun. Am. Soc. C. E. (see Item (17), Appendix) for streams in the Appalachian highlands. According to Snyder, the unit graph of a watershed depends on the time lag, $T_p$, which is defined as the time in hours from the center of mass of the rainfall that produces runoff to the time of the peak of the flood resulting from the rain. This lag, in turn, depends

upon the product of the length of the stream, $L$, in miles, multiplied by the length measured along the stream to a point opposite the center of area of the watershed, $L_c$; that is,

$$T_p = 2\ (L \times L_c)^{0.3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

A comparison of observed lags with lags computed by this formula shows a

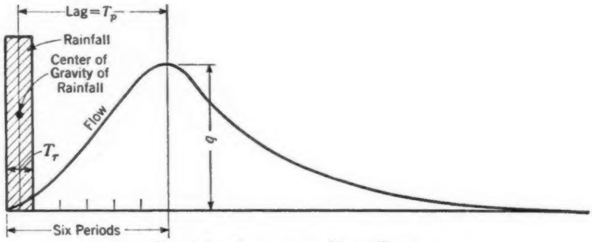



FIG. 11.—SYNTHETIC UNIT GRAPH

fairly close agreement—much closer for example than the case of lag versus area of watershed, where watersheds of different shape are compared.

The period of rainfall chosen for the unit graph is one sixth the time from the start of the runoff-producing rain to the peak of the runoff. Thus the length of this period, $T_r$, is the lag divided by 5.5, as follows:

$$T_r = \frac{T_p}{5.5} \ldots\ldots\ldots (2)$$

This relation is made clear by reference to Fig. 11. The peak flow of the unit graph in cubic feet per second per square mile ($q$) is computed by the formula:

$$q = \frac{400}{T_p} \ldots\ldots\ldots\ldots (3)$$

This is the peak flow due to 1 in. of runoff in the time, $T_r$. The coefficients in these formulas, 2 and 400, are believed to vary for differences in slope and valley storage, and are average values which give fairly consistent results for streams in the Appalachian highlands. From the curves in Fig. 12, the percentages

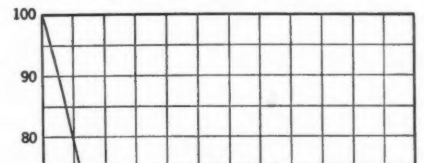



FIG. 12.—RELATION OF BASIN LAG AND DISTRIBUTION GRAPH

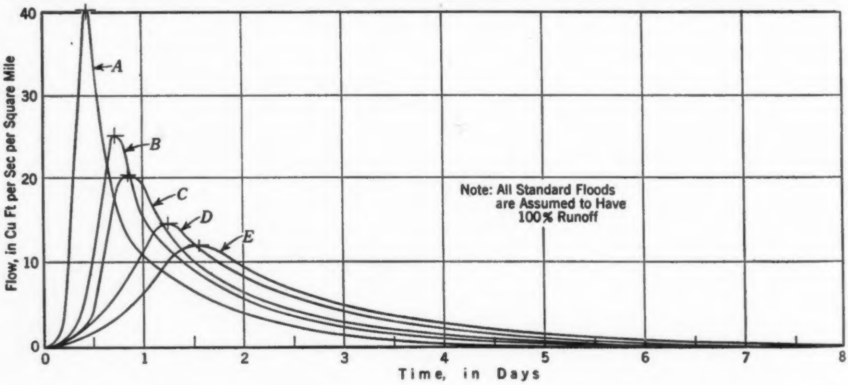
of total runoff occurring each day after the start of the rain can be read for any value of the lag. With these percentages, and the time and size of the peak computed by the formulas, it is possible to construct a unit graph in terms of

flow per square mile for any value of the product, $L \times L_c$. It is important to note that such a unit graph is independent of the area of the watershed, and depends only on the length of the stream and distance to the center of area.

For the standard watershed of length $L$ (see Fig. 9) the area is the length times the average width, taken as one-fourth the length, or $\frac{L^2}{4}$. The distance to center of area $L_c$ is $\frac{L}{2}$. Use of the average coefficients of 2 and 400 gives the standard watershed average characteristics as to slope, valley storage, etc. By means of Snyder's method unit hydrographs were computed for standard watersheds having areas of 100, 500, 1,000, 3,000 and 6,000 sq miles.

A summary of the computations of the unit graphs of the standard watershed, and the complete unit graph for the five areas of the standard sheds computed, are shown in Table 6. The graphs would be equally applicable to any watershed having the same lag.

TABLE 6.—COMPUTATION OF UNIT GRAPHS FOR STANDARD WATERSHEDS




| Item | Description | Curve $A$ | Curve $B$ | Curve $C$ | Curve $D$ | Curve $E$ |
|---|---|---|---|---|---|---|
| 1 | Area $A$, in hundreds of square miles | 1 | 5 | 10 | 30 | 60 |
| 2 | Length $L$, in miles | 20 | 44.7 | 63.3 | 110 | 155 |
| 3 | Maximum width $b$, in miles | 6.36 | 14.2 | 20.1 | 35.0 | 49.3 |
| 4 | Length $L_c$, to center of area, in miles | 10 | 22.3 | 32.6 | 55.0 | 77.5 |
| 5 | Product $L \times L_c$, in hundreds of square miles | 2 | 10 | 20 | 60 | 120 |
| 6 | Lag $T_p$, in hours | 9.9 | 15.9 | 19.7 | 27.5 | 34 |
| 7 | Time of rain $T_r$, in hours | 1.8 | 2.9 | 3.6 | 5.0 | 6.3 |
| 8 | Peak flow $q$ from a 1-in. runoff in time $T_r$, in cubic feet per second per square mile | 40.5 | 25.2 | 20.3 | 14.5 | 11.8 |

*The Standard Flood.*—The standard flood results from the standard storm occurring on a standard watershed. By means of the unit hydrographs previously developed, the three types of standard flood were determined for the five areas. The storms were read off at the intervals required by the unit hydrograph from the rainfall-duration curves, Figs. 7(*b*), 7(*c*), and 8(*b*). In the case of the small areas, with short intervals, the storm was not traced to the

full 5 days, since the rainfall at the ends was so small, relatively, that it had little effect on anything but the total length of the hydrograph. The runoff coefficient was taken at 100%.

TABLE 7.—COMPUTATION OF STANDARD FLOODS

| Item | Description | Curve *A* | Curve *B* | Curve *C* | Curve *D* | Curve *E* |
|---|---|---|---|---|---|---|
| | (a) WINTER FLOOD, OHIO AXIS | | | | | |
| 1 | Maximum rain in time $T_r$ and over area $A$, in inches | 4.0 | 3.47 | 3.20 | 2.7 | 2.35 |
| | Peak Flows, in Cubic Feet per Second per Square Mile: | | | | | |
| 2 | From maximum period of rain | 162 | 87.4 | 65.0 | 39.1 | 27.8 |
| 3 | From entire storm | 259.5 | 157.1 | 127.1 | 82.7 | 62.0 |
| 4 | Ratio, Item 3 to Item 2 | 1.6 | 1.8 | 1.96 | 2.11 | 2.23 |
| | (b) SUMMER FLOOD, OHIO AXIS | | | | | |
| 5 | Maximum rain in time $T_r$ and over area $A$, in inches | 7.2 | 6.8 | 6.2 | 5.0 | 4.3 |
| | Peak Flows, in Cubic Feet per Second per Square Mile: | | | | | |
| 6 | From maximum period of rain | 292 | 171 | 126 | 72.5 | 50.6 |
| 7 | From entire storm | 371.8 | 229.2 | 179.6 | 108.3 | 79.1 |
| 8 | Ratio, Item 7 to Item 6 | 1.27 | 1.34 | 1.42 | 1.49 | 1.58 |
| | (c) SUMMER FLOOD, ATLANTIC AXIS | | | | | |
| 9 | Maximum rain in time $T_r$ and over area $A$, in inches | 7.8 | 7.4 | 7.1 | 6.7 | 6.45 |
| | Peak Flows, in Cubic Feet per Second per Square Mile: | | | | | |
| 10 | From maximum period of rain | 316 | 186 | 144 | 97.2 | 76.0 |
| 11 | From entire storm | 435.7 | 269.3 | 215.2 | 149.7 | 122.5 |
| 12 | Ratio, Item 11 to Item 10 | 1.37 | 1.45 | 1.50 | 1.54 | 1.61 |

Table 7 shows the peak flows for the five areas computed; and, for comparison, the flow resulting from the rain of the most intense period. The ratios, Items 4, 8, and 12, indicate what an important part the short-time intensity of the storm plays and why daily rainfall records, for areas of this size, will not give consistent values of peak flow. Complete hydrographs are shown in Fig. 13 for Ohio standard floods on two watershed areas.

The peak flows are plotted against drainage area in Fig. 14, for each type of standard flood. The lines in Fig. 14 are seen to be curved concave downward. This curvature is the result of the manner in which the rainfall varies as between large and small areas. Since all three lines were derived by using the same watershed, variations in the position, slope, and curvature of these lines are due solely to the size and structure of the storm.

A common method of comparing floods from areas of different size is to assume that the flow varies as the square root of the area. A line showing a flow of $\frac{5{,}000}{\sqrt{A}}$ is shown in Fig. 14 for comparison. The standard floods vary closely as the 0.3 power of the watershed area for areas between 100 and 1,000 sq miles, and the total peak flow varies as the 0.7 power of the area. From 1,000 to 10,000 sq miles, the Atlantic summer flood continues to vary at this rate, but the winter floods vary more nearly as the 0.4 power, and the Ohio summer flood as the 0.45 power of the watershed area. These differences are

the result of differences in the structure of the storm on areas greater than 1,000 sq miles. The more nearly uniform the storm in both duration and area, the less will be the effect of watershed area on the peak flow per square mile.

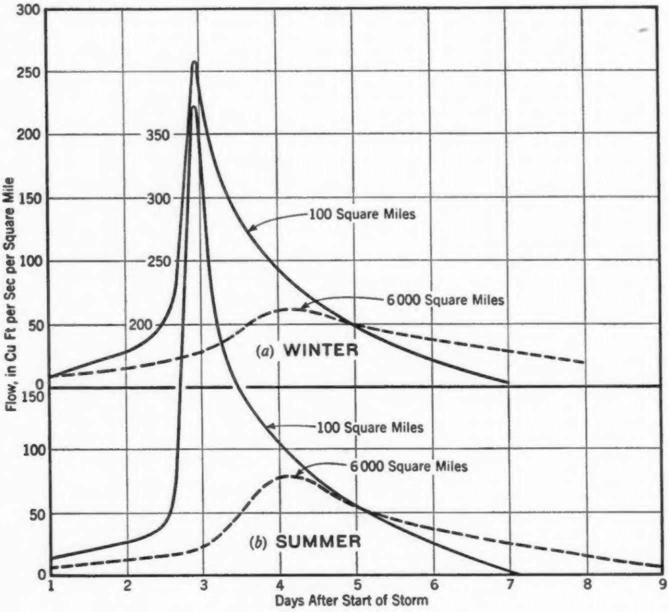



FIG. 13.—HYDROGRAPHS OF STANDARD OHIO FLOODS

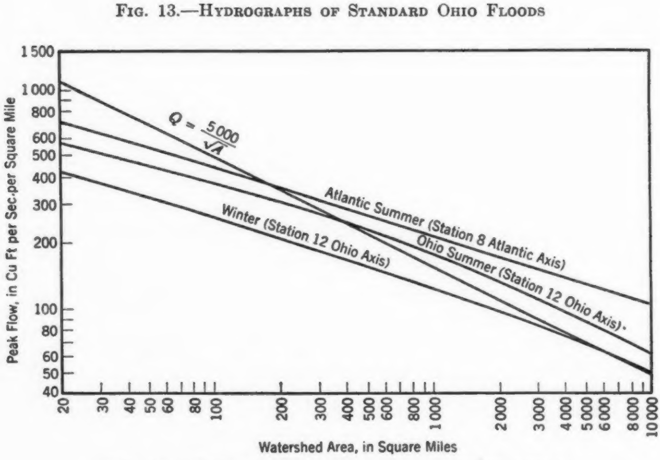



FIG. 14.—STANDARD FLOOD PEAKS FOR VARIOUS AREAS

It is evident that no single rate of variation can be applied to all seasons and areas, except as a very rough approximation. A check of the slope of the Atlantic standard flood line on Fig. 14 was made against a group of floods whose

frequency has been estimated. A list of twenty-seven 1% chance floods was given by the late Allen Hazen, M. Am. Soc. C. E. (Item (19), Appendix), for watersheds located mostly on the Atlantic Coastal Plain. They were estimated by probability methods from stream-flow records, ranging from 20 to 84 years long. These floods were corrected for shape and location of watershed to make them comparable with Atlantic summer standard floods, by a method to be presently explained. An average line through the floods cited by Hazen is roughly parallel to the standard flood line, indicating the same variation of flow with watershed area. The Atlantic standard flood, with 80% runoff, is roughly 2.5 times as large as these corrected 100-yr floods. It is about twice as large as the 1,000-yr floods as computed by Fuller's formula,

$$Q(\max) = C\, A^{0.8}\, (1 + 0.8 \log T) \left(1 + \frac{2}{A^{0.3}}\right) \qquad (4)$$

using a coefficient $C$ of 80 and time, $T$, of 1,000 years (Item (20), Appendix). In this case, too, the lines are roughly parallel. These comparisons indicate that the standard floods have about the same frequency for all sizes of drainage area. It also confirms the extreme rarity of floods of this size on any given watershed.

The foregoing discussion relates to the peak flow of the standard floods. The volume of the standard flood, in inches on the watershed for all water in excess of a given flow, expressed as a percentage of the peak flow, was computed from complete hydrographs of winter and Ohio summer storms, and is shown in Table 8. Except for the total storm, the watershed area has little effect on these volumes. The higher peaks, in flow per square mile, of the

TABLE 8.—VOLUME OF STANDARD OHIO FLOODS ABOVE VARIOUS RATES OF FLOW

| Percentage of peak flow[a] | 100-Sq-Mile Area | | | 3,000-Sq-Mile Area | | | 6,000-Sq-Mile Area | | |
|---|---|---|---|---|---|---|---|---|---|
| | Flow[b] | Volume, in inches | Percentage of total volume | Flow[b] | Volume, in inches | Percentage of total volume | Flow[b] | Volume, in inches | Percentage of total volume |
| (a) Winter | | | | | | | | | |
| 0 | 0 | 13.2 | 100 | 0 | 9.6 | 100 | 0 | 8.1 | 100 |
| 10 | 26 | 8.75 | 66.4 | 8.3 | 9.04 | 94.0 | 6.2 | 7.65 | 94.5 |
| 25 | 65 | 4.68 | 35.4 | 20.7 | 4.96 | 51.6 | 16.6 | 4.9 | 60.5 |
| 50 | 130 | 1.56 | 11.8 | 41.3 | 2.11 | 22 | 31.1 | 2.03 | 25.1 |
| 100 | 260 | 0 | 0 | 82.6 | 0 | 0 | 62.2 | 0 | 0 |
| (b) Summer | | | | | | | | | |
| 0 | 0 | 16.0 | 100 | 0 | 10.0 | 100 | 0 | 8.6 | 100 |
| 10 | 37 | 9.4 | 58.8 | 11 | 8.05 | 80.5 | 8 | 7.2 | 84.0 |
| 25 | 93 | 5.0 | 31.3 | 27 | 4.98 | 31.9 | 20 | 4.6 | 53.5 |
| 50 | 186 | 1.8 | 11.2 | 54 | 2.01 | 20.1 | 39.5 | 1.87 | 21.8 |
| 100 | 372 | 0 | 0 | 108 | 0 | 0 | 79 | 0 | 0 |

[a] Volume above zero flow equals 5-day rainfall. [b] Cubic feet per second per square mile.

smaller areas is just about offset by the longer time during which a given percentage of the peak is exceeded by the floods from the larger areas. The data indicate the storage needed, assuming perfect reservoir operation, to reduce the standard flood peak by various amounts.

## The Maximum Probable Flood

The maximum probable flood for a particular watershed is assumed to equal the flood caused by the maximum probable storm for that location, occurring so the area of maximum rainfall coincides with the watershed boundaries, and with a runoff coefficient normal for the season and quantity of rain. The severe condition of a perfect fit is assumed to offset the possibility of a larger storm, or of a higher than normal runoff coefficient, due to previous rain. It is not considered in the least probable that all of these factors would be present simultaneously. Increase in flow due to dam failures, or increase in flood heights due to ice jams or debris dams, is not considered.

Where a unit hydrograph of the watershed is available, the maximum probable flood can be computed from the standard storm, corrected for location. For other cases, and where only the peak flow is desired, it may be found by applying suitable corrections to the standard flood. Three correction factors are considered: One is for the effect of location, one for the effect of shape, and the third is the appropriate runoff coefficient. Although it would be desirable, no means were found of introducing the effect of slope and valley storage on the flood-producing capacity of a watershed. The extent to which these factors may influence the size of floods in the state may be inferred from the variations Snyder found in the two constants of his formulas. The combined effect of this variation for extreme cases is believed to be about 15% either way.

*Location Correction.*—The correction factor for location can be read from Fig. 6. It was assumed that the maximum probable storm varied according to the slope of the storm profiles on which this map was based; it follows that the floods resulting from this storm on a standard watershed will vary the same way. The Juniata Basin, above Newport, Pa., is shown in Fig. 15 as an example. From Fig. 6, the summer correction is read as 115%. The maximum probable flood at this point, therefore, would be 15% greater than the standard flood, due to the difference in location.

*Shape Correction.*—During the maximum probable flood an actual watershed will have the same rainfall as a standard watershed of the same area. It will have the same unit graph (in flow per square mile) as some other standard watershed having the same product ($L \times L_c$) and therefore the same lag. If the peak flow of the standard flood from this second watershed is corrected for the difference in the quantity of the rainfall, due to the difference in area, it will represent the maximum probable flood, at 100% runoff, from the actual watershed—subject, of course, to the location correction. The variation of rainfall with area is slightly different for different durations, but if a duration equal to the time of rain of the unit hydrograph is used, any error from this cause will be slight. The shape correction factor is defined as the ratio of the maximum probable flood to the standard flood for an area equal to the area of the actual watershed.

Referring to the example of the Juniata Basin, in Fig. 15, the value of $L \times L_c$ is $72 \times 137$, equaling 9,864. The standard watershed with this value will have an area of $\frac{9{,}864}{2} = 4{,}932$ sq miles. It is indicated in Fig. 15 by the larger ellipse. The standard Ohio summer flood from this area is 89 cu ft per sec per sq mile. The time of rain is about 6 hr, and the 6-hr rain on 4,932 sq miles is 4.5 in., and on 3,354 sq miles (the area of the Juniata) is 5 in. Thus, the Juniata flow will be $\frac{5.0}{4.5}$, or 111% of the flow from the standard shed with the same product, $L \times L_c$. This equals 111% of 89, which is 99 cu ft per

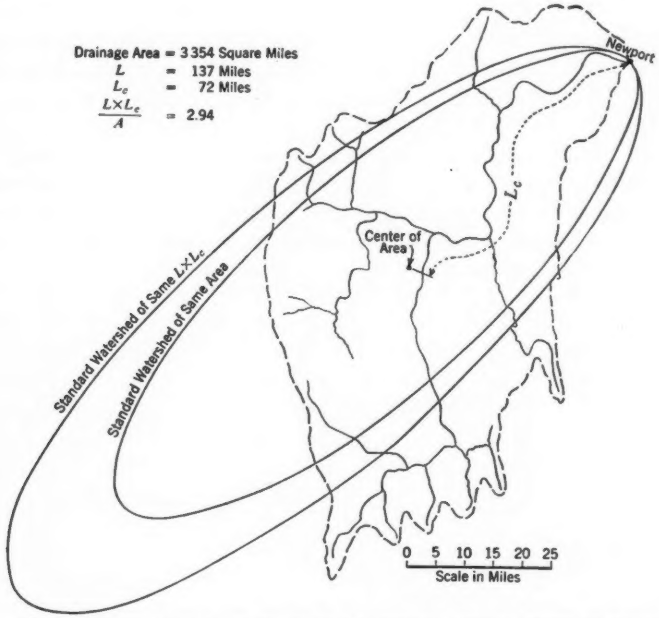



FIG. 15.—COMPARISON OF ACTUAL AND STANDARD WATERSHED; JUNIATA RIVER AT NEWPORT, PA.

sec per sq mile. The standard flood from a standard watershed, with an area equal to the Juniata (indicated by the smaller ellipse), is shown in Fig. 14 as 106 cu ft per sec per sq mile. The Juniata flood is thus $\frac{99}{106}$, equaling 93% of the standard flood, which is the desired shape-correction factor.

The variation in shape of a watershed from the standard can be measured by the ratio, $\frac{L \times L_c}{A}$. For the standard watershed, this shape ratio is 2. The shape-correction factor for a series of areas with shape ratios from 0.5 to 5 was computed, as explained, for each of the three storm types, and for various areas. The results are listed in Table 9. The range covers the variation in natural watersheds and corresponds, roughly, to a range of length-to-average-width ratios of 1 to 10. An area whose width is greater than the length will

TABLE 9.—SHAPE-CORRECTION FACTORS

| Shape ratio | (a) OHIO WINTER STORMS | | | | (b) OHIO SUMMER STORMS | | | | (c) ATLANTIC SUMMER STORMS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Drainage Areas, in Square Miles: | | | | | | | | | | | |
| | 100 | 1,000 | 6,000 | Average | 100 | 1,000 | 6,000 | Average | 100 | 1,000 | 6,000 | Average |
| 0.5 | 1.27 | 1.16 | 1.15 | 1.19 | 1.35 | 1.33 | 1.29 | 1.32 | 1.40 | 1.32 | 1.29 | 1.34 |
| 1.0 | 1.15 | 1.06 | 1.06 | 1.09 | 1.18 | 1.15 | 1.12 | 1.15 | 1.19 | 1.14 | 1.13 | 1.15 |
| 2.0 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 3.0 | 0.94 | 0.97 | 1.00 | 0.97 | 0.92 | 0.94 | 0.94 | 0.935 | 0.93 | 0.925 | 0.935 | 0.93 |
| 4.0 | 0.89 | 0.94 | 0.94 | 0.92 | 0.875 | 0.895 | 0.875 | 0.885 | 0.88 | 0.885 | 0.895 | 0.89 |
| 5.0 | 0.825 | 0.92 | 0.905 | 0.883 | 0.83 | 0.86 | 0.85 | 0.85 | 0.825 | 0.845 | 0.860 | 0.84 |

generally have two streams meeting at or near the outlet. These should be treated as two watersheds, and the results added together. Table 9 shows that variations in the shape have greater effect on the size of floods in summer than in winter. This is to be expected. Under a long, uniform storm, shape would have no effect, each square mile finally reaching a condition where it contributed the same amount to the flow. The winter storm is more uniform than the summer, as is indicated by the slope of the lines on the rainfall-duration curves; hence, shape has less effect on the size of winter floods.

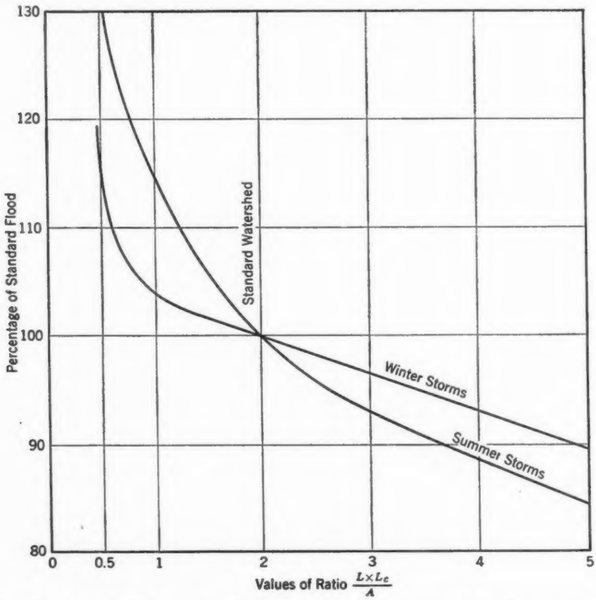



FIG. 16.—EFFECT OF WATERSHED SHAPE ON FLOODS PRODUCED BY STANDARD STORMS

Correction factors for the two types of summer storms proved to be so nearly alike that one set of values may be used for both. In Fig. 16, the average shape-correction factor for summer and winter storms, taken from

Table 9, is plotted against values of the shape ratio, $\frac{L \times L_c}{A}$. These curves can be used to find the shape-correction factor after the length of stream and length to the center of the area have been found. For this procedure, only a map of the watershed is needed. If the actual lag of the watershed can be determined from field observations on a few floods, the accuracy of the estimate will be improved. In this case, the flow per square mile will be the same as from a standard shed with the same lag, corrected for difference in rainfall on the two areas.

*Runoff Coefficient.*—The runoff coefficient is the third and last factor to apply to the standard flood to estimate the maximum probable flood from a particular watershed. As used herein, the runoff coefficient may be defined as the total surface runoff due to a particular storm, expressed as a percentage of the rainfall causing it. The runoff coefficient is not a constant, but varies with the season of the year, the size of the storm, and the character of the watershed.

Seasonal analysis of the runoff coefficient was attempted by plotting all the runoff coefficients (see Appendix) against the time of year when the flood occurred. A line defining the upper limits of the scattered points was drawn. Its shape strongly suggested the mean annual temperature curve reversed, but its position was about one month later. The highest values of the runoff coefficient thus appeared to vary inversely with the temperature of the month preceding the flood. This is probably due to the direct effect of temperature on the ground-water levels. It was decided, therefore, to use the average monthly temperature of the preceding month as a numerical expression of the season of the year. This facilitated comparison of runoff coefficients from widely separated points where the conditions in a particular month might be quite different. The runoff coefficients were classified into groups according to the mean temperature of the month preceding the flood, the first group running from 10° to 30° F, the others by 10° steps to 80° F. They were then plotted against the total inches of rain causing the flood. Two samples of these plottings are shown in Fig. 17. Lines connecting the maximum and minimum values, and smooth curves to represent the average maximum, were drawn. Points above 100%, due to snow runoff, were neglected as there was no way to separate this from the rain runoff. Although still widely scattered, the points reveal the trend of higher coefficients with lower temperatures and heavier rainfalls. The average maximum curves are shown together in Fig. 17(*c*).

The other causes for variation in the runoff coefficient, which account for the range in values for the same rainfall and temperature, are mostly associated with the particular watershed and may be assumed as constant for that shed. They include such things as the nature of the soil, kind and amount of vegetation, steepness of the slopes, presence of swamps, and the like. In general, the variations either way from the average, due to these causes, are less than the range due to season and size of rain.

Some allowance may be made for the peculiar character of the watershed if the coefficients for a few floods are available. Such values may be plotted on

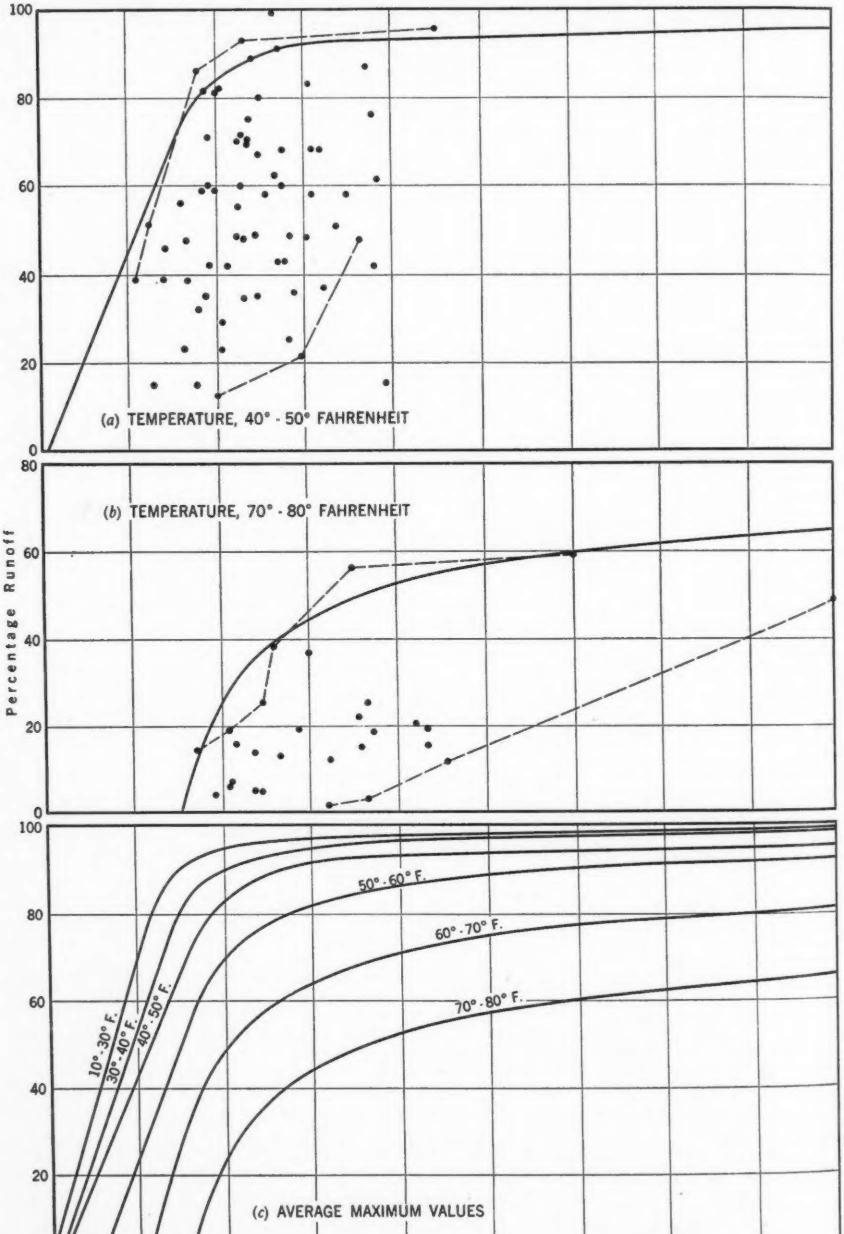



FIG. 17.—VARIATION OF THE RUNOFF COEFFICIENT WITH TOTAL RAINFALL AND WITH THE TEMPERATURE OF THE MONTH PRECEDING THE FLOOD

the figures for the appropriate amount of rain and the temperature of the preceding month. By observing the position of these points with relation to the maximum and minimum lines, the probable values of the coefficient for different seasons and larger amounts of rain may be estimated. For winter storms, the possibility of snow runoff must be considered and an allowance made for it.

In selecting a coefficient for the maximum probable storm, precision such as is required for estimating or predicting an actual flood from rainfall data at hand is meaningless, since all the conditions surrounding the future storm must be assumed. Neither is it necessary to use the highest coefficient on the record except as its size may be affected by the size of the storm. Aside from this, the storm is as likely to occur when the coefficient is low, due to a warm preceding month, as when it is high. The combination of the maximum probable storm with an extreme value of the coefficient is considered improbable. In the absence of any specific data, a value of the coefficient of 80% to 90% for summer and 120% for winter would seem to be safe for the purpose.

*Estimating the Maximum Probable Flood.*—The process of determining the maximum probable flood may be summarized thus:

$$\text{Maximum probable flood} = \text{standard flood} \times \text{location correction} \times \text{shape correction} \times \text{runoff coefficient} \times \text{drainage area} \quad (5)$$

To return to the example of the Juniata at Newport, the maximum probable summer and winter flood peaks would be computed as shown in Table 10 (see

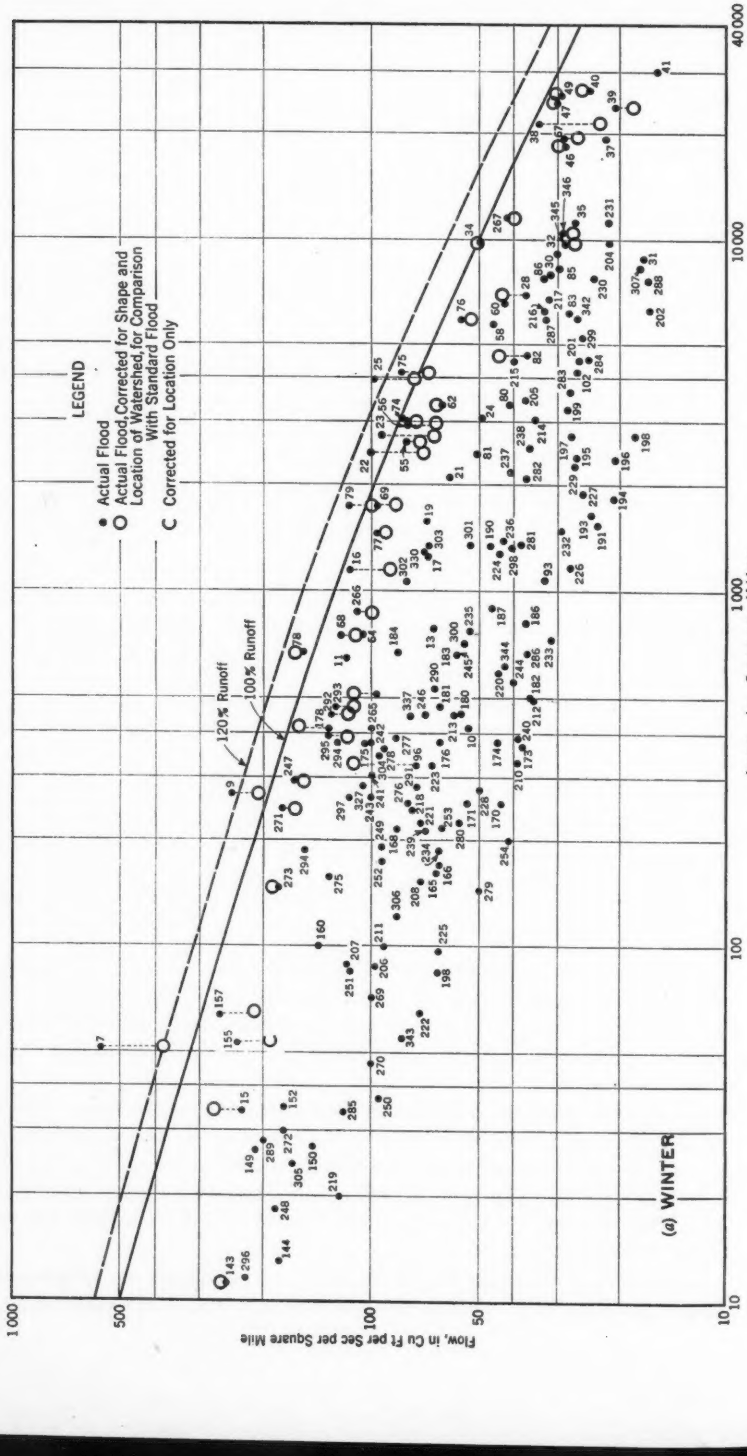
TABLE 10.—COMPUTATION OF MAXIMUM PROBABLE SUMMER AND WINTER FLOODS; JUNIATA RIVER AT NEWPORT, PA.

| Item | Description | See Fig. | Summer | Winter |
|---|---|---|---|---|
| 1 | Standard flood from 3,354 sq miles, in cubic feet per second per square mile | 14 | 106 | 80 |
| 2 | Location correction (percentages) | 6 | 115 | 98 |
| 3 | Shape correction (percentages) for a shape ratio of 2.94 | 16 | 93 | 97 |
| 4 | Assumed runoff coefficient (percentages) | .... | 90 | 120 |
| | Maximum Probable Flood: | | | |
| 5 | Cubic feet per second per square mile | .... | 102 | 91 |
| 6 | Total, in thousands of cubic feet per second | .... | 340 | 306 |

Fig. 15). The maximum floods of record were: Summer (June, 1889), 240,000 cu ft per sec; and winter (March, 1936), 215,000 cu ft per sec. The same corrections can be applied to the standard flood volumes in Table 8 to obtain the volume of the maximum probable flood at Newport above various channel capacities.

In this case, the location and shape corrections have only a minor effect on the size of the flood as compared with possible errors in the size of the standard storm and flood. For the average case, their use is of more value as an assurance that these factors have been taken care of than to increase the absolute accuracy of the estimate. Their possible combined effect within the state ranges from 70% to 160%.

In a strict sense, something might be added to the floods as determined in Table 10 for the base flow—that is, the flow which would have occurred in the


LEGEND
• Actual Flood
O Actual Flood, Corrected for Shape and Location of Watershed, for Comparison With Standard Flood
C Corrected for Location Only
120% Runoff
100% Runoff
(a) WINTER
Area, in Square Miles
Flow, in Cu Ft per Sec per Square Mile

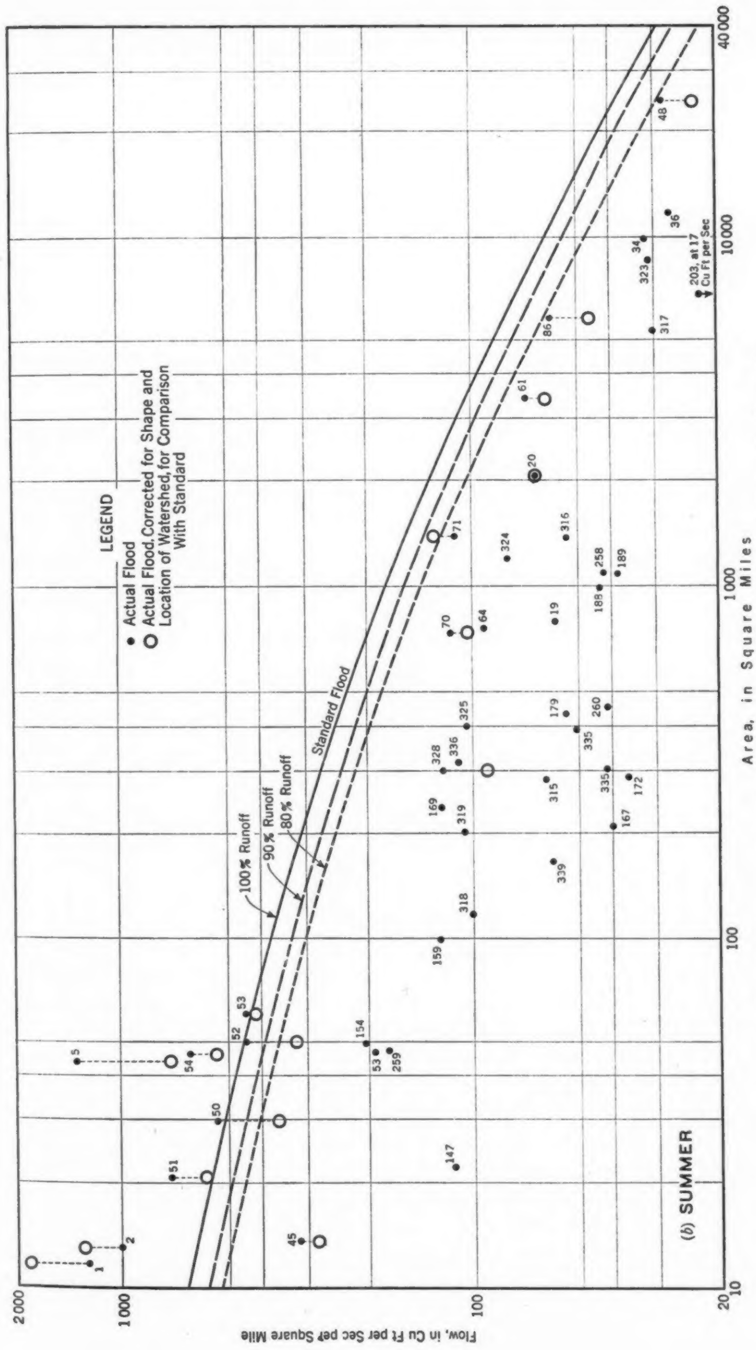



FIG. 18.—COMPARISON OF STANDARD AND ACTUAL FLOODS; STATION 12, OHIO AXIS

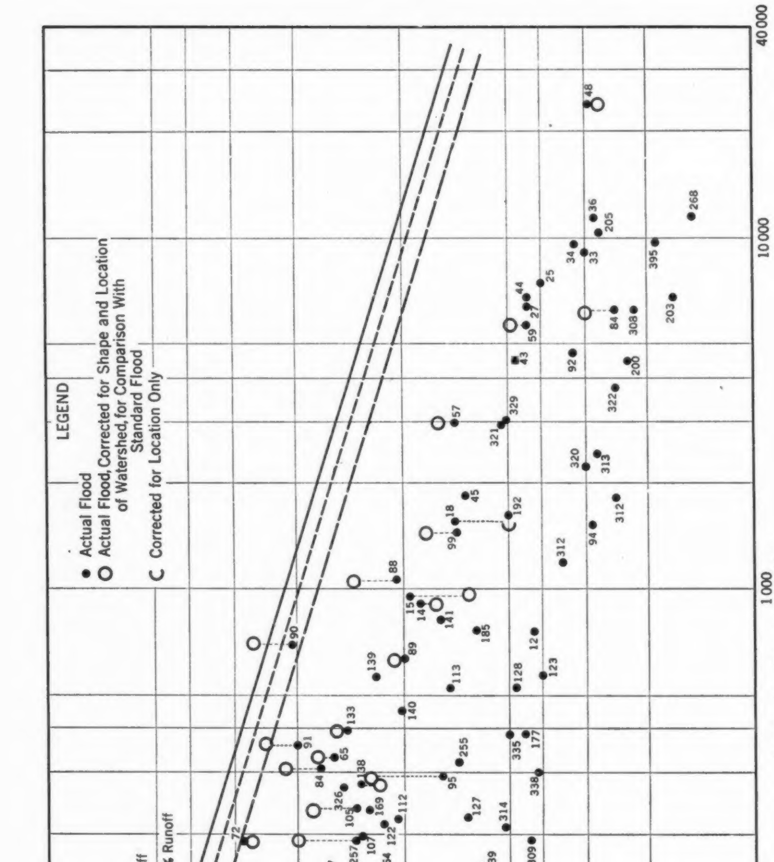



FIG. 19.—COMPARISON OF STANDARD AND ACTUAL FLOODS; STATION 8, ATLANTIC AXIS; SUMMER

stream had there been no rain. In comparison with floods as large as the maximum probable flood, any normal base flood would be insignificant. Occasionally, a flood occurs superposed on a large base flow from a preceding storm, but such an event is considered to be offset by the unlikelihood of its coinciding with the maximum value of the other factors that affect flood size.

## Comparison with Flood Records

By applying the three correction factors to an actual recorded flood peak, an estimate can be made of the flood that would have been produced by the same storm had it occurred on a standard shed at station 12 or 8. A comparison of this corrected actual flood with the standard flood for the same drainage area will give a measure of the relative size of the storm, and of the flood resulting from it. A list of all the large floods in Pennsylvania and the adjacent states was compiled (see Appendix) and the floods separated into the three types—winter, Atlantic summer, and Ohio summer. In Figs. 18 and 19 these floods are shown as peak flow per square mile against watershed area. The three standard floods were drawn on the figures for comparison, together with lines representing 80% and 90% for summer runoff and 120% for winter runoff. Those floods that exceeded the standard flood, or approached it closely, were then corrected for location and shape of watershed to make them comparable with the standard floods. For a few floods, as indicated, maps could not be secured to make the shape correction. The runoff coefficient is unknown for most of them, and no correction was made for it. Figs. 18 and 19 show the extent to which the maximum probable flood, as derived from storm data, checks the past record. There is a tendency for the standard flood to be exceeded on the small areas (less than 100 sq miles) to about the same extent as a few short-duration rainfalls exceeded the standard storm (see Figs. 7($a$) and 8($a$)).

For the larger areas the Atlantic standard flood (Fig. 19) exceeds the actual floods by a considerable margin. Considering only the flood points, it appears that a steeper line, perhaps one represented by the formula

$$Q = \frac{K}{\sqrt{A}} \qquad (6)$$

would fit the data better. Because of the nature of the data plotted, however, it is believed that such a curve would not represent floods having the same probability of occurrence as between large and small areas. The record for the Atlantic coastal region includes very few basins of 10,000 sq miles, or more, in area. At the other end of the scale, there are hundreds of watersheds of 50 to 100 sq miles in area. Although all of these are not gaged, when one of them produces a notable flood some estimate is generally made of it and it finds its way into the records. Thus, the Chester Creek flood of 1843, which has not been approached anywhere in this region in the succeeding hundred years, is included and would probably be included had it occurred on any one of a hundred streams of comparable size. This flood obviously represents a very different degree of probability in respect to occurrence on a particular designated watershed than do the recorded floods for areas around 10,000 sq miles.

The Ohio standard floods, where the actual flood records for large areas are more numerous, are in better agreement.

Both the standard flood lines, derived entirely from rainfall data, and the records of actual floods, as corrected, appear to indicate an upper limit to flood size which will rarely be exceeded.

## Conclusion

It is not the writer's intention to present another flood formula or curve of maximum floods. Neither is the maximum probable flood offered as a criterion to be used blindly for design of flood-control projects or spillways, since factors of cost, benefits from protection, and damage done by failure are all involved in such decisions. The methods are described and the data displayed with the hope of making clearer the flood-producing mechanism; of giving quantitative significance, however approximate, to the relative importance of the factors that influence the size of floods; and finally, to serve as a guide to judgment in selecting the size of flood to be provided for.

## Acknowledgment

Most of the work on which this paper is based was done for the Pennsylvania Flood Control Bureau. The writer wishes to acknowledge the help of G. Gale Dixon, M. Am. Soc. C. E., former chief of the Bureau, for valuable suggestions and criticisms; of Mr. Snyder for advice concerning the application of his synthetic unit graphs; and of C. L. Rice and the late F. M. Felker, Jun. Am. Soc. C. E., for assisting with the computations involved.

# APPENDIX

## Source and Scope of Data

(1) The data on the storm paths were taken from U. S. Weather Bureau records, in which maps are available showing the path of the low-pressure areas since about 1890. Synoptic weather maps, which would throw considerably more light on storm behavior, are a development of recent years, and would not supply a sufficiently long record.

(2) The data on storms used on the storm profiles come mainly from "Storm Rainfall in Eastern United States," published in 1936 by the Miami Conservancy District. This report covered all the storms in the United States east of the 104th meridian which had a rainfall of 6 in. in 3 days at five or more rainfall stations for the period 1892 to 1933. The total number was 280 storms, and included 3 before 1892. Of these, the time-area-depth curves for the 73 most important storms are given in the book. Two more storms, Nos. 4 and 265, for which such curves were not given, but which occurred near Pennsylvania, were estimated and included. The following four other storms occurring in or near the state between 1933 and 1937 were added: July 7–8, 1935, in New York; September 4–5, 1935, in Maryland; March, 1936, in Pennsylvania and New England; and January, 1937, in Tennessee and Kentucky. No attempt was made to bring the report up to date for the entire

United States; but with these additions, the data include the largest storms in and near the state for the 45 years 1892–1937.

(3) The "Miami Report" was also the source of data on the highest rainfall at a single station in each 2° quadrangle, and of the 1-day to 5-day rainfall having a 100-yr frequency at stations 8 and 12.

(4) For shorter durations, data were taken from "Rainfall-Intensity-Frequency Data," by David L. Yarnell, *Miscellaneous Publication No. 204*, U. S. Dept. of Agriculture.

(5) Other data on short-duration rainfall were taken from a report made by Arthur E. Morgan, M. Am. Soc. C. E., to the Pennsylvania Water Supply Commission, in July, 1921. This contained some very old records—notably, Catskill, N. Y., in 1817, and Chester, Pa., in 1843. Some of the recent hourly rainfall records were taken from U. S. Weather Bureau records.

(6) Data on the runoff coefficient were secured from the following sources:

(7) The *Journal* of the Boston Society of Civil Engineers for September, 1930, gives runoff coefficients for the Ware, Swift, Westfield, Nepaug, Farmington, and Waschuset rivers in New England and the Miami River in Ohio. A total of about fifty values for watersheds, ranging from 50 to 2,500 sq miles, are given.

(8) *Water Supply Paper No. 772*, U. S. Geological Survey, contains runoff coefficients for the Delaware, Muskingum, Susquehanna, and Wabash rivers. A total of twenty-five values is given for areas ranging from 3,000 to 8,000 sq miles.

(9) The U. S. Corps of Engineers developed runoff coefficients for streams in and near the Upper Susquehanna Basin, the Chemung, Cohocton, Unadilla, and Chenango rivers. Some 200 values were used from this list. Unlike the remainder of the data, the base flow was not excluded in computing the coefficient. For this reason, values for the smaller rains, where the base flow might have been a large part of the total, were excluded and nearly all the records used were for rains of 2 in. or more.

(10) About fifteen values of the coefficient, computed by the writer for various streams in the state, were also used.

(11) All the data represent the runoff coefficient for one particular flood, based on the rain that caused it. Rainfalls of various intensities and durations are naturally included.

(12) The flood records plotted in Figs. 18 and 19 were taken from "Flood Flow Characteristics," by C. S. Jarvis, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985, *et seq.* This contains a very complete list of floods up to 1926. It was supplemented and brought up to date with data from the following:

(13) *Journal*, Boston Soc. of Engrs., September, 1930.

(14) "New York State Flood of 1935," *Water Supply Paper No. 773E*, U. S. Geological Survey.

(15) "The Floods of March 1936," *Water Supply Papers Nos. 798, 799, and 800*, U. S. Geological Survey.

(16) The Water Supply Papers, in addition to the particular flood discussed, give values for the highest previous flood and thus serve to bring Mr. Jarvis' comprehensive record up to date. As far as possible, these data were compared and checked with a list of the largest floods in Pennsylvania, compiled by J. W. Mangan, M. Am. Soc. C. E., of the Pennsylvania Division of Hydrography, in which a number of the previously accepted values for peak flow have been corrected.

Other sources cited are:

(17) *Transactions*, Am. Geophysical Union, Part 1, 1938, pp. 447–454.

(18) *Proceedings*, Am. Soc. C. E., June, 1938, p. 1296.

(19) "Flood Flows," by Allen Hazen, John Wiley and Sons, Inc., New York, N. Y., 1930, pp. 63 and 64.

(20) *Transactions*, Am. Soc. C. E., Vol. LXXVII, December, 1914, p. 564.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

# SUPERSTRUCTURE OF THEME BUILDING OF NEW YORK WORLD'S FAIR

By Shortridge Hardesty,[1] M. Am. Soc. C. E., and Alfred Hedefine,[2] Assoc. M. Am. Soc. C. E.

Synopsis

There are discussed in this paper the unusual engineering problems involved in the design and construction of the Theme Center of the 1939 New York World's Fair. The points of special interest are the spherical steel framework of the Perisphere, the triangular steel tower of the Trylon, and the determination of the wind loads to which these two structures may be subjected.

## Introduction

The Theme Building for the 1939 New York World's Fair, the general features of which are shown in Fig. 1, is composed of four integral parts—the Perisphere, the Trylon, the Bridge, and the Helicline. The Perisphere is the hollow white sphere 180 ft in diameter—as tall as a 16-story building. The Trylon is the needle-like structure—a slender, triangular pyramid taller than the Washington Monument, with its axis 200 ft southeast of the center of the Perisphere. The Bridge is the connecting link between the Perisphere and Trylon, and carries escalator units and walkways. The Helicline is the inclined circular ramp, 950 ft long, which serves as the exit from the group.

The Perisphere, the Trylon, and the Bridge are of steel-frame construction, covered for the most part with a thin stucco covering of a special type, and supported on concrete foundations that rest on creosoted timber piles. The different structures presented individual design problems, some of which were very unusual. The latter statement is particularly true of the Perisphere, which involved the construction of a complete spherical framework, subjected to both vertical loads and transverse wind forces. For its design, the application of the theories of spherical shells was studied thoroughly; but it was eventually found possible, by means of careful reasoning and the visualization of the stress action throughout the frame, to employ the methods applied in the analysis of usual structural frames, in spite of the novelty of the arrangement of the members involved.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **January 15, 1941.**

[1] Cons. Engr. (Waddell & Hardesty), New York, N. Y.

[2] Associate Engr., Waddell & Hardesty, New York, N. Y.

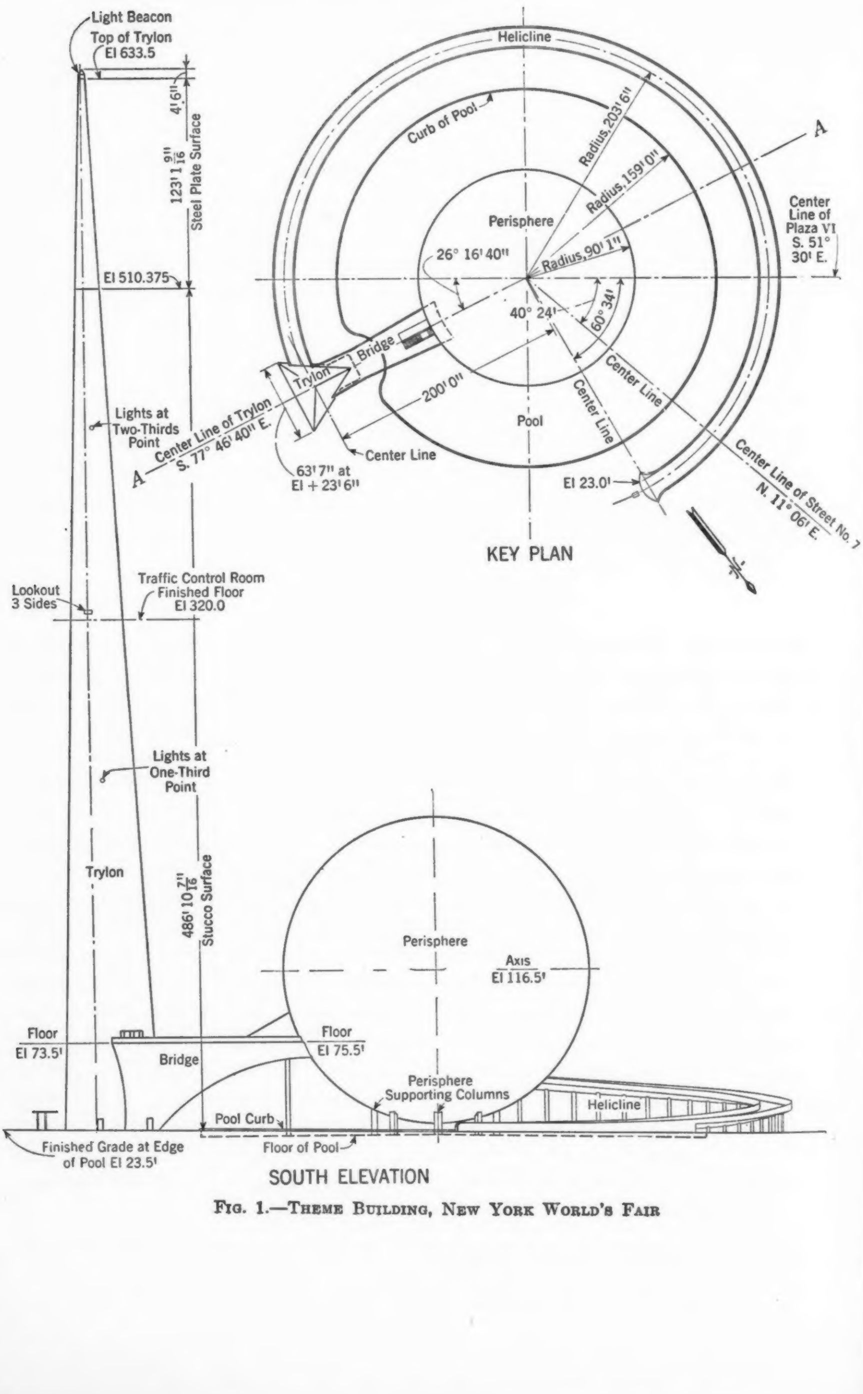



Fig. 1.—Theme Building, New York World's Fair

The Trylon was unique in that it involved the use of a triangular braced tower so slender that the design was controlled by the wind forces rather than the dead and live loads.

Interesting data regarding the four units of the Theme Building are as follows:

| Item No. | Description | Value |
|---|---|---|
| | *Perisphere* | |
| 1 | Finished outside diameter, in feet | 179.17 |
| 2 | Finished height above ground, in feet | 182.58 |
| | Diameter of Steel Frame, in Feet: | |
| 3 | Outer diameter | 178.58 |
| 4 | Inner diameter | 162.25 |
| | Meridian Trusses: | |
| 5 | Number | 32 |
| | Depths, in Feet, at: | |
| 6 | Bottom | 11.0 |
| 7 | Equator | 8.0 |
| 8 | Top | 5.0 |
| | Girt Trusses: | |
| 9 | Number | 15 |
| 10 | Depths, in feet | 5 to 10.5 |
| | Ring Girder at Columns (Feet): | |
| 11 | Diameter | 72.0 |
| 12 | Depth | 7.71 |
| 13 | Width | 2.01 |
| | Columns: | |
| 14 | Number | 8 |
| 15 | Height, in feet | 16.75 |
| 16 | Section, in feet | 2.65 by 1.88 |
| 17 | Number of pieces | 6,600 |
| 18 | Number of shop rivets | 150,000 |
| 19 | Number of field rivets | 100,000 |
| 20 | Weight of steel, in pounds | 4,300,000 |
| | Covering: | |
| 21 | Total area of covering, in square feet | 100,851 |
| 22 | Layers of $\frac{1}{2}$-in. gypsum board (dampproofed between layers and nailed to wooden struts, 2 in. by 4 in.) | 2 |
| 23 | Thickness of stucco, in inches (special magnesite compound) | 0.25 |
| | Moving Platforms: | |
| 24 | Number (an upper and a lower) | 2 |
| | Dimensions, in Feet: | |
| 25 | Height above ground (upper and lower) | 64, 52 |
| 26 | Height above Theme exhibit (upper and lower) | 30, 18 |
| 27 | Diameter to outer edge (upper and lower) | 113, 103 |
| 28 | Width | 6 |
| 29 | Distance from inner wall of sphere | 12 |
| 30 | Weight, each platform, in pounds | 200,000 |
| 31 | Speed, in feet per minute | 60 |
| 32 | Riding time, in minutes | 5.5 |
| 33 | Estimated spectator load, in pounds | 120,000 |

| Item No. | Description | Value |
|---|---|---|
| | Dimensions of Reflecting Pool, in Feet: | |
| 34 | Diameter | 318 |
| 35 | Depth | 1.5 to 3.5 |
| | Foundations: | |
| 36 | Length of piles (528 creosoted Douglas fir), in feet | 95 to 104 |
| | Footing Dimensions, in Feet: | |
| 37 | Diameter | 71 |
| 38 | Width | 14.5 |
| 39 | Depth | 3.5 |
| | Ring Wall Dimensions, in Feet: | |
| 40 | Diameter | 72 |
| 41 | Width | 4.5 |
| 42 | Depth | 7.75 |
| | Dead Load, in Pounds: | |
| 43 | Structural framework | 4,300,000 |
| 44 | Outer shell | 720,000 |
| 45 | Inner shell, insulation, walkways, exhibit, and equipment | 1,900,000 |
| 46 | Moving platforms | 400,000 |
| 47 | Subtotal (items 43–46) | 7,320,000 |
| 48 | Live load, in pounds | 480,000 |
| 49 | Snow load, in pounds | 370,000 |
| | Total Load, in Pounds: | |
| 50 | 8 columns (items 47–49) | 8,170,000 |
| 51 | 1 column | 1,022,000 |
| | Wind Loads, in Pounds per Column: | |
| 52 | Transferred load | 270,000 |
| 53 | Shear | 82,000 |
| 54 | Moment at bottom of column due to wind loads, in pound-feet | 656,000 |
| | *Trylon* | |
| | Dimensions of Superstructure, in Feet: | |
| 55 | Height above ground | 610.0 |
| 56 | Base (length of each side) | 63.58 |
| 57 | Top (length of each side) | 2.58 |
| 58 | Distance of observation room (for traffic control) above ground | 296.5 |
| 59 | Weight of steel, in pounds | 1,830,000 |
| | Covering: | |
| 60 | Total area, in square feet | 60,540 |
| 61 | Year 1939 | |
| | Layers of gypsum board (see item 22) | 1 |
| | Outer surface, magnesite compound stucco | .... |
| 62 | Year 1940 | |
| | Lower 100 ft stucco | .... |
| | 390 ft of plywood painted white | .... |
| 63 | Height of steel plate construction, in feet, from the top | 124 |
| | Foundation Dimensions, in Feet: | |
| 64 | Length of piles (513 creosoted Douglas fir) | 95 to 104 |
| | Hexagonal Bases: | |
| 65 | Thickness of two 40-ft bases | 7.00 |

| Item No. | Description | Value |
|---|---|---|
| 66 | Thickness of one 46-ft base | 7.75 |
| | Dead Load, in Pounds: | |
| 67 | Structural framework | 1,830,000 |
| 68 | Covering | 270,000 |
| 69 | Walls, ceilings, enclosures, anchors | 1,200,000 |
| 70 | Subtotal (items 67–69) | 3,300,000 |
| 71 | Live load, in pounds | 110,000 |
| | Total Load, in Pounds: | |
| 72 | 3 columns (items 70–71) | 3,410,000 |
| 73 | 1 column | 1,137,000 |
| | Wind Load per Column, in Pounds: | |
| 74 | Downward | 2,580,000 |
| 75 | Uplift | 2,870,000 |
| | Maximum Load per Column, in Pounds: | |
| 76 | Downward | 3,717,000 |
| 77 | Uplift | 1,770,000 |
| | *Bridge* | |
| | Dimensions of Superstructure, in Feet: | |
| 78 | Length | 98.5 |
| 79 | Height at Perisphere and at Trylon | 52, 50 |
| 80 | Radius of soffit line | 113.72 |
| 81 | Length of supporting columns at the Perisphere | 49 |
| 82 | Weight of steel, in pounds | 290,000 |
| 83 | Covering, magnesite compound stucco | .... |
| | Moving Stairways (Two, an Upper and a Lower): | |
| 84 | Length, in feet (upper and lower) | 120, 96 |
| 85 | Vertical height, in feet (upper and lower) | 60, 48 |
| 86 | Width, each, in feet | 2 |
| 87 | Angle of rise, in degrees | 30 |
| 88 | Speed, in feet per minute | 90 |
| 89 | Capacity in persons per hour, each | 8,000 |
| 90 | Power (electricity) | .... |
| | Foundations: | |
| 91 | Number of piles (creosoted Douglas fir) | 32 |
| 92 | Base dimensions, in feet | 23.5 by 10 by 5 |
| | *Helicline* | |
| 93 | Grade (percentage) | 5.5 |
| | Dimensions, in Feet: | |
| 94 | Length | 950 |
| 95 | Width | 18 |
| 96 | Radius | 203.5 |
| | Pipe Column Supports: | |
| 97 | Height | 18 to 50 |
| 98 | Diameter | 1.5 to 2.5 |
| 99 | Deck material (timber with stainless steel soffit and cork flooring) | .... |
| 100 | Total cost of Theme Buildings, in dollars | 2,000,000 |

## Preliminary Design Considerations: Wind Loads

Before the engineers began the design of the Perisphere and Trylon, the architects had determined the proportions of the two objects and the general

arrangement of the passageways, escalators, moving platforms, and exhibits, and had decided that the Perisphere would be over a circular pool, supported on a row of eight columns arranged in a circle of 72-ft diameter, or on a continuous ring of the same diameter. The column supports were finally chosen, partly for architectural reasons, and partly because wind tunnel tests (described subsequently herein) showed that the wind pressure against the Perisphere would be less for the column supports than for the continuous ring. After the framework of the Perisphere and Trylon had been laid out, the details of the passageways, platforms, etc., were determined, and the resulting loads on the frameworks computed. By assuming the type of cover to be used, the vertical loads, both dead and live, on all parts of the two structures could be calculated.

For the determination of the dead loads, the cover was assumed to be cement stucco, 1 in. thick, weighing 17 lb per sq ft. When a light cover, of a patented magnesite compound, was finally selected in 1939, the Perisphere design was not affected; but the reduction of load on the Trylon required that additional weight be provided to overcome the increased uplift from wind loads. This result was secured by supporting the concrete floor of the Trylon at ground level on heavy concrete beams in which the bottom bracing members of the Trylon framework were embedded.

The foundation materials at the site of the Theme Center consist of about 30 ft of cinders and about 45 ft of mud resting on firm material. In line with other heavy foundations on the Fair grounds, piles driven to the firm material were used; and in order to make it safe to leave the structures standing after the closing of the Fair, if desired, creosoted piles were adopted.

It was obvious that the design of the Perisphere and Trylon would be affected by wind loads; and since structures of these types and sizes had not been constructed before, and information relative to the probable wind action on them was scarce and unsatisfactory, it was decided that a wind-tunnel investigation should be made. This investigation consisted of a series of tests performed under the supervision of Alexander Klemin (Chairman of the Department, Daniel Guggenheim School of Aeronautics, College of Engineering, New York University, New York, N. Y.).[3] The Perisphere model was 2 ft in diameter and was raised 0.08 ft above the ground board; the Trylon model was 6.75 ft high, with a triangular base 0.66 ft on a side. All tests were made in a 9-ft wind tunnel at the laboratory.

The test program included drag and pressure tests on the Trylon and Perisphere, alone and in combination with each other, in various positions relative to the direction of the wind. In order that the "ground effects" experienced by the structures as actually built might be reproduced as closely as possible, the tests of the Perisphere, with and without the Trylon, were made with a "ground board" in the tunnel. The kinematic viscosity was assumed to be 0.000154 ft squared per sec, and the Reynolds number was taken equal to the product of the wind speed in feet per second by the diameter in feet by the constant 6,500 for the case of the Perisphere, and by the length in feet of

[3] "Aerodynamics of the Perisphere and Trylon at World's Fair," by A. Klemin, E. B. Schaefer, and J. G. Beerer, Jr., *Proceedings*, Am. Soc. C. E., May, 1938, p. 887.

a side of the base by the same constant for the case of the Trylon. The largest values of the Reynolds number for the wind-tunnel tests, for the cases of the Trylon alone, the Perisphere alone, and the Perisphere and Trylon in combination were 378,000, 572,000, and 1,340,000, respectively.

Table 1 indicates the values of the pressures on the Perisphere and Trylon for the various conditions tested. It shows the pressure, in pounds per square

TABLE 1.—PRESSURES IN POUNDS PER SQUARE FOOT OF PROJECTED AREA

| Item No. | Description | Formula[a] | Shape factor | Pressure[a] for: | |
|---|---|---|---|---|---|
| | | | | $V = 100$ | $V = 90$ |
| 1 | Velocity head | $0.00256\ V^2$ | 1.0 | 25.6 | 20.8 |
| 2 | Large, square, flat plate, normal to wind | $0.0032\ V^2$ | 1.25 | 32 | 26 |
| 3 | Prism 1 by 1 by 3 | $0.0038\ V^2$ | 1.48 | 38 | 30.8 |
| 4 | Infinitely long prism or flat plate | $0.0050\ V^2$ | 1.95 | 50 | 40.5 |
| 5 | Trylon on the ground: Flat side into wind | $0.0037\ V^2$ | 1.45 | 37 | 30 |
| 6 | Edge into wind | $0.0023\ V^2$ | 0.90 | 23 | 18.6 |
| 7 | Perisphere alone, on ring wall, on the ground | $0.0015\ V^2$ | 0.59 | 15 | 12.2 |
| 8 | Perisphere on ring wall, with Trylon, on ground | $0.0018\ V^2$ | 0.70 | 18 | 14.6 |
| 9 | Perisphere alone, on columns, on ground | $0.0013\ V^2$ | 0.51 | 13 | 10.5 |
| 10 | Perisphere on columns, with Trylon, on ground | $0.0016\ V^2$ | 0.62 | 16 | 13 |
| 11 | Perisphere in free air | $0.0005\ V^2$ | 0.195 | 5 | 4 |

[a] $V$ = velocity in miles per hour.

foot of projected area, for any velocity in miles per hour, together with the values for 100-mile and 90-mile velocities. For reference, the table also includes values for velocity head, and for pressures on large square flat plates, on prisms three times as long as wide, and on indefinitely long prisms or flat plates.

Current specifications usually provide that bridges and buildings shall be designed for a maximum wind pressure of 30 lb per sq ft. By referring to Table 1 it will be noted that this pressure corresponds to the value for a large square flat plate at a wind velocity of 100 miles per hr, and also to that for a 1 by 1 by 3 prism in a 90-mile wind.

The "shape factors" determined from the tests on the Trylon and Perisphere, expressed as the ratio of the average wind pressure to the velocity head, are worthy of note. The free Perisphere in an air flow of 90 miles per hr gave an average pressure of 4 lb per sq ft on the projected area, corresponding to a shape factor of 0.195; whereas, when the Perisphere was subject to the same air flow in the presence of the ground, supporting columns, and Trylon, the shape factor increased to 0.62, corresponding to an average pressure of 13 lb per sq ft on the projected area, or an increase in the wind force of three and a quarter times. This phenomenon, caused by the ground and structure effects, indicates the importance of the tests from the structural design point of view.

It is of interest to note that, in the test of the Trylon with a flat side into the wind, there were pressure decreases, or suctions, of sufficient force on those faces away from the direction of the wind to result in a force greater than the velocity pressure times the projected area, corresponding to a shape factor of 1.45. It might also be mentioned that tests with the Trylon directly in front

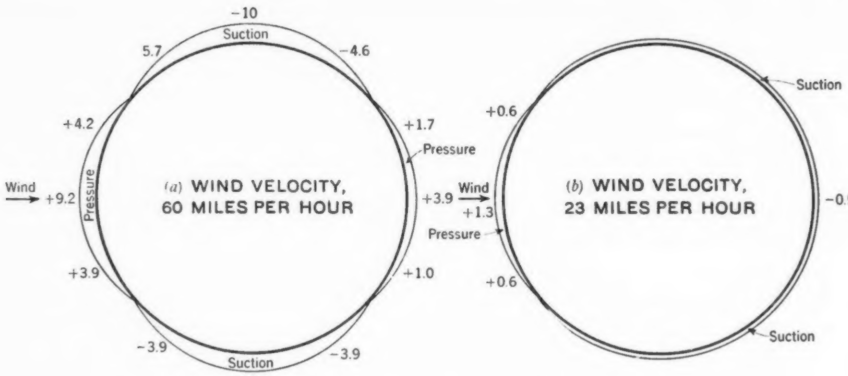



Fig. 2.—Wind Pressures on a Sphere in Free Air (Diameter 7.94 Ft)

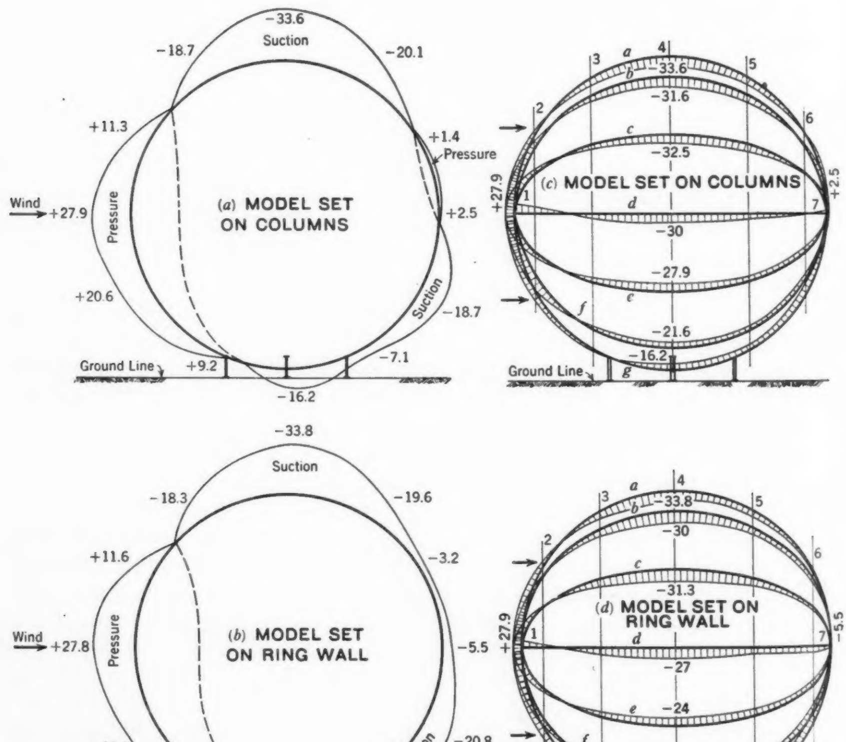



Fig. 3.—Pressures on a Free Perisphere Model Due to a Wind Velocity of 100 Miles per Hr

of the Perisphere, blocking the path of the wind, gave a net pressure on the Perisphere of practically nothing.

The values in Table 1 indicate the horizontal resultant of the pressures created by the flow of wind. The drag tests on the Perisphere showed, in addition to the horizontal force, a considerable uplift, amounting to 70% of the horizontal force in the case of the Perisphere alone on ring wall adjacent to the ground, and 60% of the horizontal force in the case of the Perisphere alone on columns adjacent to the ground.

Fig. 2 shows the center meridian pressures, in pounds per square foot, for the supercritical and subcritical states, as determined by O. Flachsbart,[4] at the Göttingen Aerodynamics Institute, on a wood sphere 24.2 cm (9.52 in.) in diameter in free air—that is, with no ground board. Figs. 3(*a*) and 3(*c*) indicate the pressures, in pounds per square foot, on the center meridian of the Perisphere for a wind velocity of 100 miles per hr, for the case of the sphere on columns. The broken lines are isobars of zero pressure. Figs. 3(*b*) and 3(*d*) show graphically the pressure contours for the same model, set on a ring wall. The shaded portion on the outside of a great circle indicates positive pressure, whereas that on the inside of the circle represents a negative pressure, or suction.

The diagrams of Fig. 4 represent the pressures resulting from a wind velocity of 100 miles per hr, based on the various cases tested by Professor Klemin[3] (see Fig. 5). Case I is for that of the Perisphere alone on columns. In this case it is of interest to note that a turbulence was created in the flow of air tailing off the rear of the sphere, thereby forming a back pressure acting against the direction of the wind. Here, as in all cases, the maximum positive pressure occurs on the surface exposed directly to the wind, and the maximum negative pressure on the areas at right angles to the direction of the wind. Case II is for the condition of the Perisphere on columns with the Trylon at its side (with respect to the direction of the wind). This gives essentially the same family of pressure curves as in Case I. The back pressure of Case I, however, has now changed to a slight suction. Case III is for the Perisphere on a ring wall. For Case IV there is the same combination as Case II except that the Perisphere is on a ring wall instead of columns. In both Case III and Case IV there is a considerable build-up of suction at the rear of the sphere, which increases measurably the net force tending to push the Perisphere off of its foundations. Case V is that of the Perisphere on the ring wall with the Trylon directly in front of it with respect to the wind direction. The net result of the pressures for this last case, for all practical purposes, was negligible.

The pressures plotted for each case are those along the seven semi-arcs of great circles which pass through the front and rear poles and whose planes make angles above and below the equator of 0°, 30°, 60°, and 90°. The pressure points are located by increments of 30° azimuth angles through 180° from front to rear. The net wind force acting on the Perisphere for each case tested can be obtained by graphic integration of these pressure curves.

The pressures given in the various diagrams represent the net pressures

[4] "Recent Researches on the Air Resistance of Spheres," by O. Flachsbart, *Technical Memorandum No. 475*, National Advisory Committee for Aeronautics, 1928.

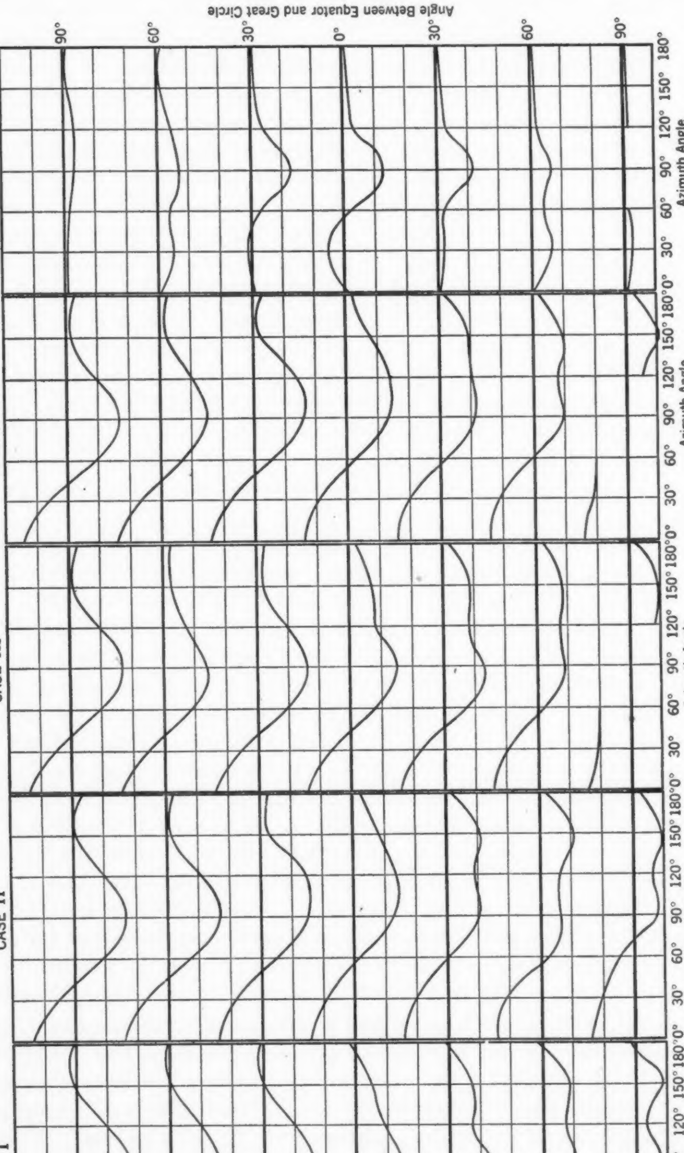



FIG. 4.—PRESSURES ON A PERISPHERE MODEL, WITH TRYLON IN VARIOUS POSITIONS; WIND VELOCITY 100 MILES PER HR

existing on the Perisphere covering at any point when the air pressure within the Perisphere is the same as that in the moving air on the outside. Since the covering is nearly impervious, however, the pressure within may at times be practically that of still air, which is greater than that of moving air by the amount of velocity head. At such times the net pressures on the covering, instead of varying from the velocity head at the portion facing the wind to a suction about one third greater at a point 180° thereto, will vary from about zero at the first point to a suction two and one third times the velocity head at the second point. The suction acting on the cover, therefore, may be as great as 60 lb per sq ft.

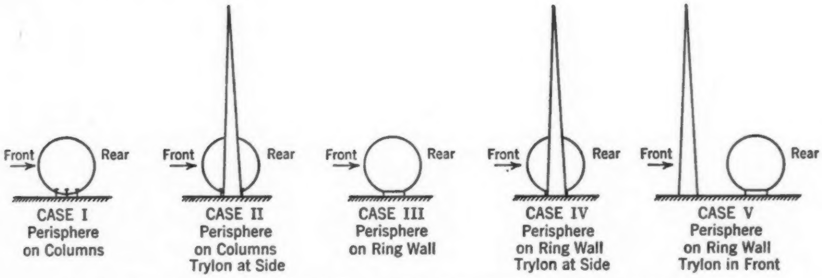



FIG. 5.—POSITIONS OF STRUCTURES WITH RESPECT TO WIND, IN TESTS OF FIG. 4

The curves shown in Fig. 6 present an interesting comparison of wind pressures acting on the Perisphere under various conditions.

As has been previously indicated, there is a large variation in the wind pressures acting over the surfaces of the Perisphere and Trylon. For the Trylon there is positive pressure over the face or faces exposed to the wind, and suction on those away from the direction of the wind, with the maximum pressure approximately equal to the velocity head, and the resultant force acting at a point one third of the way up from the ground surface. In the case of the Perisphere, a bulb of positive pressure occurs over the portion of the surface directly facing the wind, and a high negative pressure or suction occurs over the surface area at 90° to the direction of the wind, reducing to a minimum pressure at a point 180° from the wind direction.

In order to arrive at pressure values for the design of the Perisphere, consideration had to be given to the fact that the presence of the Bridge increased appreciably the forces on the Perisphere itself. The Bridge also causes a slight increase in the wind pressure on the Trylon, but this effect is inappreciable, as the force is applied close to the bottom of the Trylon.

The symmetrical area of suction around the complete Perisphere produces tension in that group of meridian trusses, with very little distortion of the girts. The pressure on the front of the Perisphere, the resultant of which is inclined upward, and the suction on the back, the resultant of which is inclined downward, form a set of forces tending to roll the sphere as a whole.

As the maximum wind velocity in the New York area approximates 90 miles per hr, it was decided to base the design loads on that velocity, with

higher values for checking stability against overturning. Therefore, the following loads, in pounds per square foot, were adopted:

| Description | For designing members | For determining stability and anchorage |
|---|---|---|
| Trylon with flat side into wind........ | 30 | 50 |
| Trylon with edge into wind........... | 20 | 33 |
| Perisphere on columns............... | 15 | 25 |

Since the completion of the Perisphere and Trylon, they have been subjected to several heavy winds and found to be satisfactory. During the hurri-

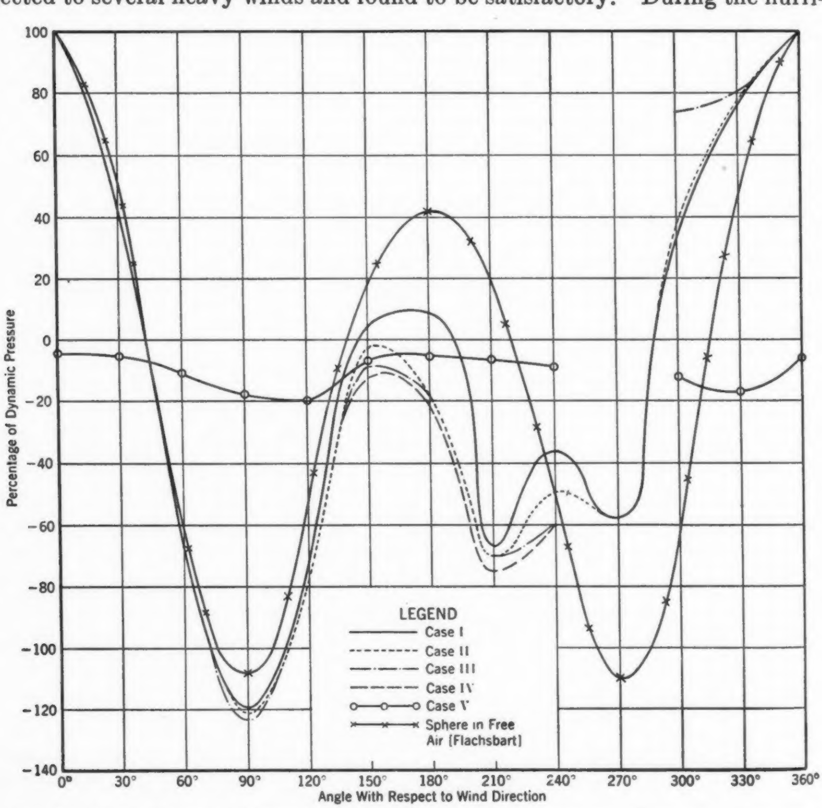



FIG. 6.—PRESSURES ON A MERIDIAN CIRCLE EXPRESSED AS PERCENTAGES OF DYNAMIC PRESSURE

cane of September, 1938, a wind velocity of about 75 miles per hr was reached at the site of the Fair. At this time the steel framework of the Trylon, including the steel plate section at the top (but no other outside covering), was completed, and the steel frame of the Perisphere was finished but free of covering or scaffolding. During the storm it was observed that the top of the Trylon moved about 6 in. in either direction from the vertical, which movement compares with the computed movement under full design wind load of about 3 ft.

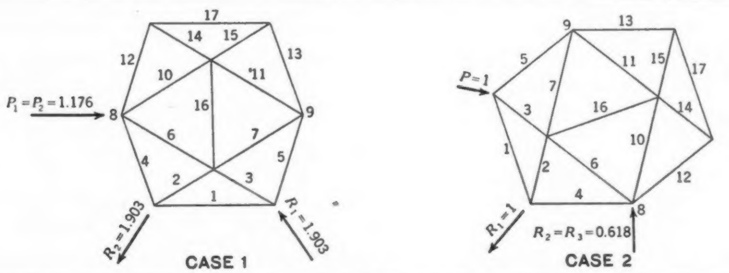
A second observation was made on the Trylon during a subsequent wind which attained a velocity of 45 miles per hr. At that time three fourths of the surface had been covered and a complete scaffolding system shrouded the entire structure. The observed movement at an angle of about 45° to the direction of the wind was about 7 in. on either side of the vertical.

No measurements have been made on the movements of the Perisphere, but after its completion an observer standing at the junction of the Bridge and the Perisphere, with one foot on each structure, noted that only the faintest tremor could be detected during a 45-mile wind.

The framework of the Perisphere was planned to consist of a series of great-circle meridian trusses lying in vertical planes and a series of small-circle horizontal girts, with diagonal bracing in each of the trapezoids formed by the meridians and girts. Analysis showed that this framework, when loaded with symmetrical vertical loads, would have its members subjected to direct stresses of compression and tension, plus secondary bending effects that could be computed, and would transfer the loads directly to the supports. The effect of horizontal forces or unbalanced vertical loads could not be visualized readily, and it was considered desirable to study some other form of framework that could be analyzed directly for such forces and loads. The regular polyhedron having the largest number of faces—the regular icosahedron, consisting of thirty members and twenty triangular faces—was therefore studied.

The icosahedron frame is shown in Table 2. Two loading conditions were considered. In Case 1, the icosahedron is supported at the two ends of one

TABLE 2.—STRESSES IN THE ICOSAHEDRON FRAME




| Number | Stress Influence Values | | Number | Stress Influence Values | | Number | Stress Influence Values | | Number | Stress Influence Values | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | | Case 1 | Case 2 | | Case 1 | Case 2 | | Case 1 | Case 2 |
| 1 | −0.138 | −0.072 | 6 | −1.100 | −0.578 | 11 | −0.292 | −0.153 | 16 | +0.361 | +0.190 |
| 2 | +1.421 | +0.746 | 7 | +0.516 | +0.271 | 12 | −0.043 | −0.023 | 17 | −0.138 | −0.072 |
| 3 | −1.198 | −0.629 | 8 | +0.642 | +0.337 | 13 | −0.043 | −0.023 | .... | .... | .... |
| 4 | +0.458 | +0.240 | 9 | +0.362 | +0.190 | 14 | +0.112 | +0.059 | .... | .... | .... |
| 5 | −0.543 | −0.285 | 10 | −0.292 | −0.153 | 15 | +0.112 | +0.059 | .... | .... | .... |

side, and subjected to two horizontal radial forces at mid-height; and in Case 2, it is supported on the three corners of one face, and subjected to a single inclined force. The resulting stresses, which are determinate, are given in the

table. By analogy, it was determined that the spherical framework would also transmit transverse forces to the supports by means of direct stresses in the frame members plus secondary bending effects, as in the case of vertical loads.

The icosahedron analogy also afforded a means of estimating the wind stresses in the diagonal bracing. For the Perisphere, the horizontal force of 2 $P$ for Case 1 would be about 330,000 lb, which would produce a compression of about 200,000 lb in the diagonals leading down to the leeward support, and a tension of about 220,000 lb in the diagonals leading down to the windward support. Four diagonal members were available to take each of these forces.

## Types of Construction Considered

*Perisphere.*—Preliminary studies for the Perisphere showed that the following types of construction merited consideration: (1) Steel truss framework, separate outer shell; (2) steel beam framework, separate outer shell; (3) steel truss framework, welded steel shell; (4) steel beam framework, welded steel shell; and (5) reinforced concrete shell. The following description of these five types is adapted from reports submitted to the Construction Department of the Fair on April 12, 1937:

Each design provided a sphere of 180 ft outside diameter, carried on eight columns resting on concrete foundations. The columns were to be on a circle of 36-ft radius, the bottom of the sphere being placed 3 ft above the surface of the pool. It was assumed that the outer surface was to be watertight, of accurate outline, as smooth as practicable, and preferably free from visible joints. Provision was made in each design for supporting a lighter inner shell about 164 ft in diameter, with its center 3 ft above that of the outside spherical surface; heat insulation and acoustic treatment; two moving balconies for spectators; two emergency walkways between the outer and inner shells; access platforms to the escalators; a footbridge to the Trylon; exhibits; and equipment. It was assumed that the Bridge would not impose any loads on the Perisphere.

For each of the five layouts, the entire sphere was assumed to be a unit frame or shell, without hinges at any point. The four steel designs involved frameworks of meridians and horizontal ring girts, and were designed by making successive adjustments in assumed sections, unit stresses, deflections, and total stresses. The reinforced concrete layout involved a shell, unstiffened except between the columns, and was designed by means of theories developed for spherical shells.

In the first layout, the framework consisted of trussed members from 5 ft to 11 ft deep, occupying the entire space between the outer and inner shells except at the bottom. There were thirty-two main meridians, spaced about 18 ft apart at the equator. In general, the ring girts were about 16.5 ft on centers. The panels between meridians and rings were braced with X-bracing of single-angle members. The outer shell, which was not intended to take stress, was assumed to be carried by vertical purlins supported by the girts. The inner shell was to be carried by light members supported on the inner flanges of the meridians and girts. The meridians were carried by a circular

box girder of 36-ft radius, resting on eight supporting columns. The columns were assumed fixed at their upper ends by the ring girder and the meridians. The design assumed riveted construction throughout, although welded work could be used in certain parts if preferred.

The second layout was similar to the first, except that the members in the upper part of the sphere were curved wide-flanged beams instead of trussed members. Hangers would be required to support the inner shell in this part of the sphere.

The third layout involved a framework similar to that of the first layout, but considerably lighter, with a butt-welded steel outer shell that participated in carrying stress. The diagonal bracing was omitted, as the shell performed its function. The shell was $\frac{5}{16}$ in. thick for the upper parts of the sphere, and reached a maximum thickness of $\frac{7}{16}$ in. in the lower part.

The fourth layout was similar to the third, except that the framework in the upper part consisted of curved beams, as in the case of the second layout.

In the reinforced concrete shell layout, the thicknesses as designed were 3 in. at the top, 4 in. at the equator, and 2 ft 6 in. at the supports. Concrete stresses were low, averaging 200 lb per sq in. except at the supports, where large bending moments occurred. It was assumed that the thin parts of the shell would be of gunite, and the thicker parts of poured concrete. There was ample precedent in European practice for the shell thicknesses adopted, but the requirements of American practice might have called for some modifications; and the conditions at the support were sufficiently unusual to have justified the making of model tests for this part if the reinforced concrete design had been adopted.

The estimated costs of the superstructure and the loads on the foundations for the five layouts were as follows:

| Layout | Description | Cost | Total load on foundations, in pounds |
|---|---|---|---|
| 1 | Steel truss framework, separate outer shell | $453,000 | 9,300,000 |
| 2 | Steel beam framework, separate outer shell | 401,000 | 9,000,000 |
| 3 | Steel truss framework, welded steel shell | 554,000 | 7,400,000 |
| 4 | Steel beam framework, welded steel shell | 518,000 | 7,200,000 |
| 5 | Reinforced concrete shell | 425,000 | 15,100,000 |

The costs of the foundations were to be added in order to determine the comparative total costs.

In comparing the various designs, the following points had to be considered, in addition to the costs:

(1) *Steel Truss Framework, Separate Outer Shell.*—The framework could be fabricated and erected readily by methods in common use, and presented no difficulties other than those arising from the novel shape of the structure. It would provide a stiff, sturdy structure of accurate outline. The outer shell

probably would call for unusual care in construction, if a satisfactory surface were to be obtained. There would be some possibility of objectionable cracks developing as a result of temperature changes or participation in stress action.

(2) *Steel Beam Framework, Separate Outer Shell.*—The comments in item (1) would apply in general to this design. The fabrication should be cheaper than for the first layout, and the erection somewhat more difficult and possibly more expensive. The resulting structure would be entirely satisfactory, although not quite as stiff as in the case of the first layout.

(3) *Steel Truss Framework, Welded Steel Shell.*—The framework members could be fabricated and erected readily. The construction of the shell would call for unusual care in order to secure a satisfactory surface and avoid locked-up temperature stresses. The major difficulty would be to secure even joints; but it was believed that this could be overcome by accurate forming of the plates, and properly controlled welding procedure, with small welding rods. It might be desirable to have tests made to demonstrate the possibilities in this respect. The resulting structure would be somewhat stiffer than that of the first layout; and the shell would be certain to be watertight and free from cracks.

(4) *Steel Beam Framework, Welded Steel Shell.*—The comments in items (1) to (3) would apply in general to item (4), with the modifications noted in connection with layout 2.

(5) *Reinforced Concrete Shell.*—This layout probably would present more construction problems than would the four steel designs, items (1) to (4), but it was believed that they could be solved successfully by companies experienced in concrete construction. The falsework would have to be designed and constructed carefully, in order to secure correct outlines. Accurate workmanship would be required in order to procure a satisfactory outer surface, but this requirement might apply equally to layouts (1) and (2).

After consideration of the construction problems and the relative substructure costs for the various types, it was decided to adopt layout 1, which was found to be no more expensive than the other types, and could be expected to be free from difficult, unforeseen problems that might prove embarrassing. Actual construction has indicated that a correct choice was made.

*Trylon.*—For the Trylon, four designs were considered, as follows:

| Design | Description | Estimated cost of superstructure |
|---|---|---|
| 1 | Braced steel tower construction, surface of thin steel plate | $231,000 |
| 2 | Braced steel tower construction, gunite or stucco surface | 231,000 |
| 3 | Flat plate construction, consisting of welded steel plate properly stiffened | 247,000 |
| 4 | Reinforced concrete construction | 185,000 |

For designs 1 and 2, the top quarter was to be of flat plate construction similar to design 3.

After taking into consideration the construction problems and the total costs, including foundations, design 2 was adopted.

## Geometry and Framework

The outside surface of the completed Perisphere is a true spherical surface, 180 ft in diameter. The inner surface is fundamentally a sphere of 162-ft diameter, with its center 3 ft above the center of the outside sphere; but this surface is modified in the lower part to suit the exhibition and the moving platforms.

The entire Perisphere frame is a unit, of riveted construction throughout, without hinges at any point. The framework consists of curved truss members from 5 ft to 11 ft in depth. There are 32 meridians, each a half of a great circle, intersecting at the top and the bottom, and 15 horizontal girts from 7 ft to 16 ft on centers. The trapezoidal panels formed by the meridians and girts are braced, in the outer surface only, with X-bracing of single-angle members. The intersections of the girts with the meridians lie on small circles; but each section of girt between meridians lies in a great circle. As a result, all meridian and girt members are plane, with all outside chords bent to a

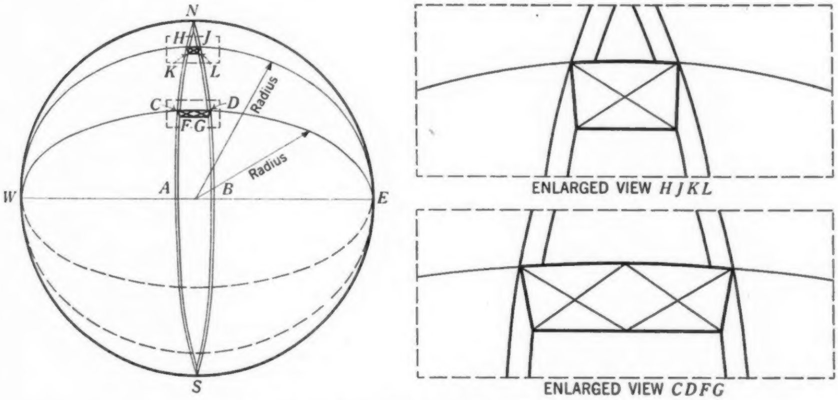
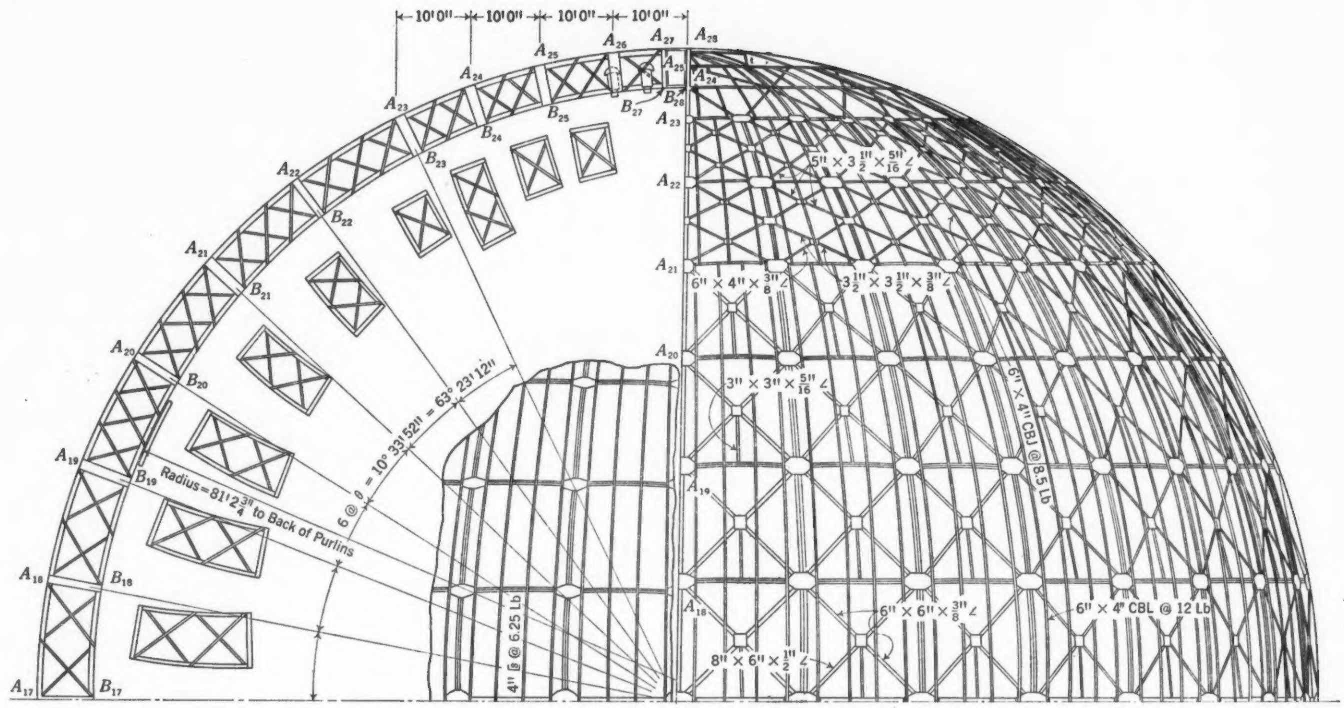



Fig. 7.—Description of Girt Trusses

uniform radius of 89.29 ft, except for the bottom members, and all inside chords bent to a uniform radius of 81.29 ft. All gusset plates are dished to spherical surfaces. This arrangement of members was evolved by the designers; it is believed to be unique in spherical space structures, and conducive to accurate and satisfactory fabrication. For the girts, it avoided the use of conical girders that would have resulted if each section of girt had been bent to a small circle. The outer sphere was exact in regard to both meridians and girts, whereas the inner sphere was exact in regard to the meridians, but involved slight approximations in regard to the girts.

Fig. 7 will serve to clarify the foregoing description of the girt trusses. If $W\ C\ D\ E$ is a great-circle plane, the intersection of this plane with two meridian trusses such as $N\ A\ S$ and $N\ B\ S$ forms the girt truss section $C\ D\ G\ F$. The outer chord, $C\ D$, is an arc of the great circle $W\ C\ D\ E$ of the outer sphere. The inner chord of the girt sections, $F\ G$, is bent to the radius of 81 ft $3\frac{1}{2}$ in., a great circle arc of the inner sphere. A girt section between any two adjacent



10'0"
10'0"
10'0"
10'0"
A17
A18
A19
A20
A21
A22
A23
A24
A25
A26
A27
A28
B17
B18
B19
B20
B21
B22
B23
B24
B25
B27
B28
Radius = 81' 2 3/4" to Back of Purlins
6 @ θ = 10° 33' 52" = 63° 23' 12"
4" [ @ 6.25 Lb
5" × 3 1/2" × 5/16" ∠
6" × 4" × 3/8" ∠
3 1/2" × 3 1/2" × 3/8" ∠
3" × 3" × 5/16" ∠
6" × 4" CBJ @ 8.5 Lb
6" × 6" × 3/8" ∠
6" × 4" CBL @ 12 Lb
8" × 6" × 1/2" ∠

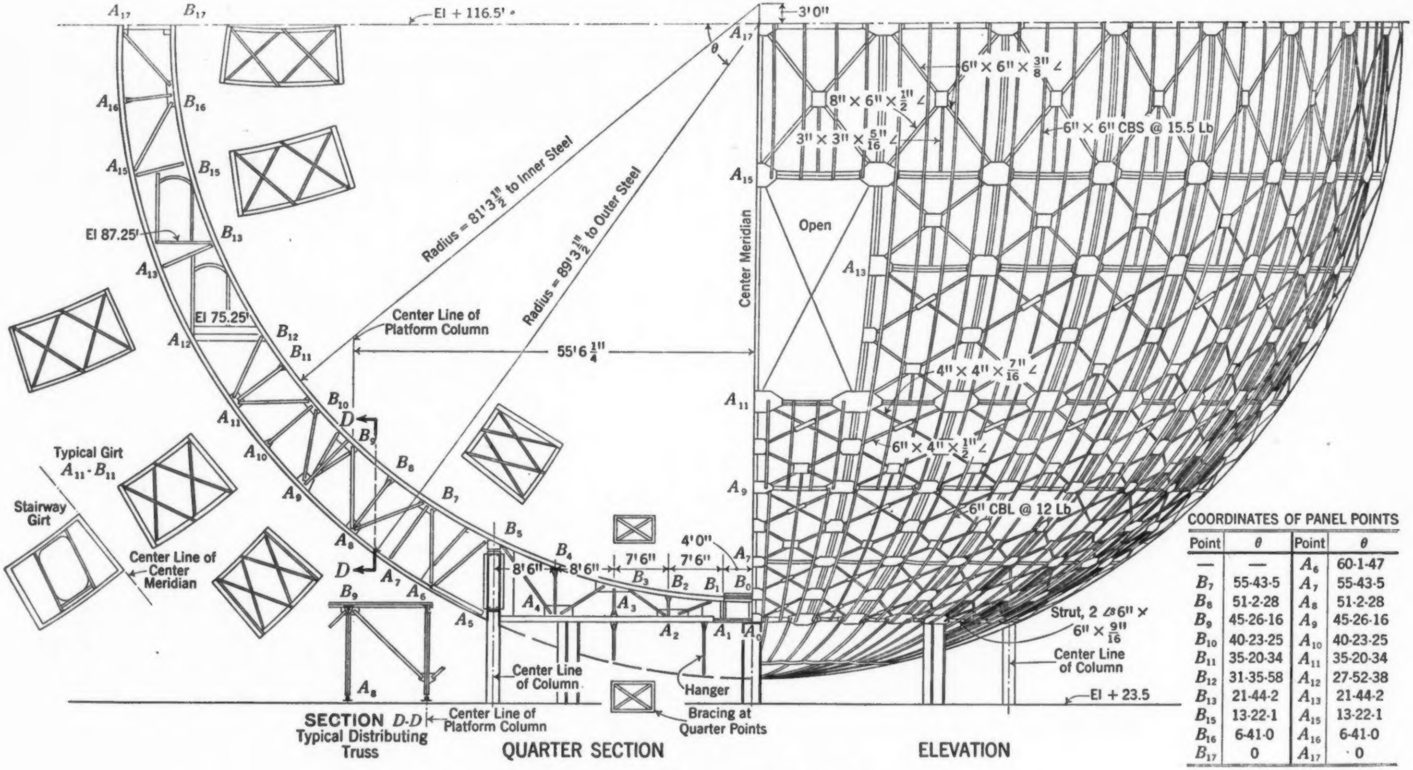



Fig. 8.—Steel Skeleton of Perisphere

COORDINATES OF PANEL POINTS

| Point | $\theta$ | Point | $\theta$ |
|---|---|---|---|
| — | — | $A_6$ | 60-1-47 |
| $B_7$ | 55-43-5 | $A_7$ | 55-43-5 |
| $B_8$ | 51-2-28 | $A_8$ | 51-2-28 |
| $B_9$ | 45-26-16 | $A_9$ | 45-26-16 |
| $B_{10}$ | 40-23-25 | $A_{10}$ | 40-23-25 |
| $B_{11}$ | 35-20-34 | $A_{11}$ | 35-20-34 |
| $B_{12}$ | 31-35-58 | $A_{12}$ | 27-52-38 |
| $B_{13}$ | 21-44-2 | $A_{13}$ | 21-44-2 |
| $B_{15}$ | 13-22-1 | $A_{15}$ | 13-22-1 |
| $B_{16}$ | 6-41-0 | $A_{16}$ | 6-41-0 |
| $B_{17}$ | 0 | $A_{17}$ | 0 |

meridians lies in a plane, that of the outer great circle; and since the inner sphere is eccentric with respect to the outer sphere, it follows that the outstanding legs of the angles of the inner chord of the girt section will project, by small amounts, beyond or within the inner spherical surface. This slight deviation is readily taken up by a minor adjustment in the inside purlin system which ultimately supports the inner covering.

The adjacent girt section to the one just described will be contained in another great-circle plane with its chords as arcs of great circles. For any one girt truss, all points of intersection with the meridians, or corresponding points between meridians, lie on a circle of latitude. This type of construction, as can be seen readily in elevation, gives a scalloping effect to the girts—more pronounced as the distance, up or down, from the equator increases. The equatorial girt, being both a great circle and a circle of latitude, appears in elevation as a straight line. In plan the equatorial girt chords lie in a circle; but all others (and increasingly so as they approach the poles) will appear as nearly straight lines.

The upper hemisphere of the Perisphere, as shown in Fig. 8, is made, in the main, of a top center drum and a complete system of meridians and girts. Eight main meridian trusses begin at the drum (see Fig. 9). At a distance of 20 ft from the vertical axis the number of meridians is doubled; and then at a distance of 40 ft from the vertical axis the meridians are increased to thirty-two in number; and from this latter point on the number is unchanged. There are ten girts, including the equatorial ring, in the upper half of the Perisphere.

The lower hemisphere is composed essentially of thirty-two meridians, six girts, a ring girder, a bottom center drum, and eight columns. All thirty-two meridian trusses intersect the ring girder and pass on to butt against the bottom drum.

The meridian trusses are carried on a curved box girder 72 ft in diameter, resting on eight steel columns. Within the ring girder the top flanges of the meridians continue at the same radius, whereas the bottom flanges are horizontal. The columns frame into the ring girder at their tops, whereas at the bottom they rock radially but are fixed tangentially. The rocking surfaces are provided to reduce temperature stresses.

The outer covering is carried on a series of vertical I-beam purlins, bent to great circles (meridians) and supported on the outer flanges of the girts. The inner shell is carried on a similar purlin system of light vertical channels fastened to the inner flanges of the girts. It is interesting to note that the vertical system of purlins showed a net saving of about 80 tons of steel over a horizontal system.

The Trylon can be classified as a tower structure—a very unusual tetrahedron. This shaft rises from an equilateral triangular base measuring 63.58 ft on a side to 610 ft in the air, and terminates in a top measuring 2.58 ft on a side. A light beacon 4.5 ft high surmounts the structure, making a total height of 614 ft 6 in. The structural framework for the lower 490 ft (see Fig. 1) consists of three columns with three planes of double X-bracing with transverse sheets and vertical hangers. The upper 124 ft consists of stiffened steel plates, which also form the outer surface of the Trylon. The columns for the lower

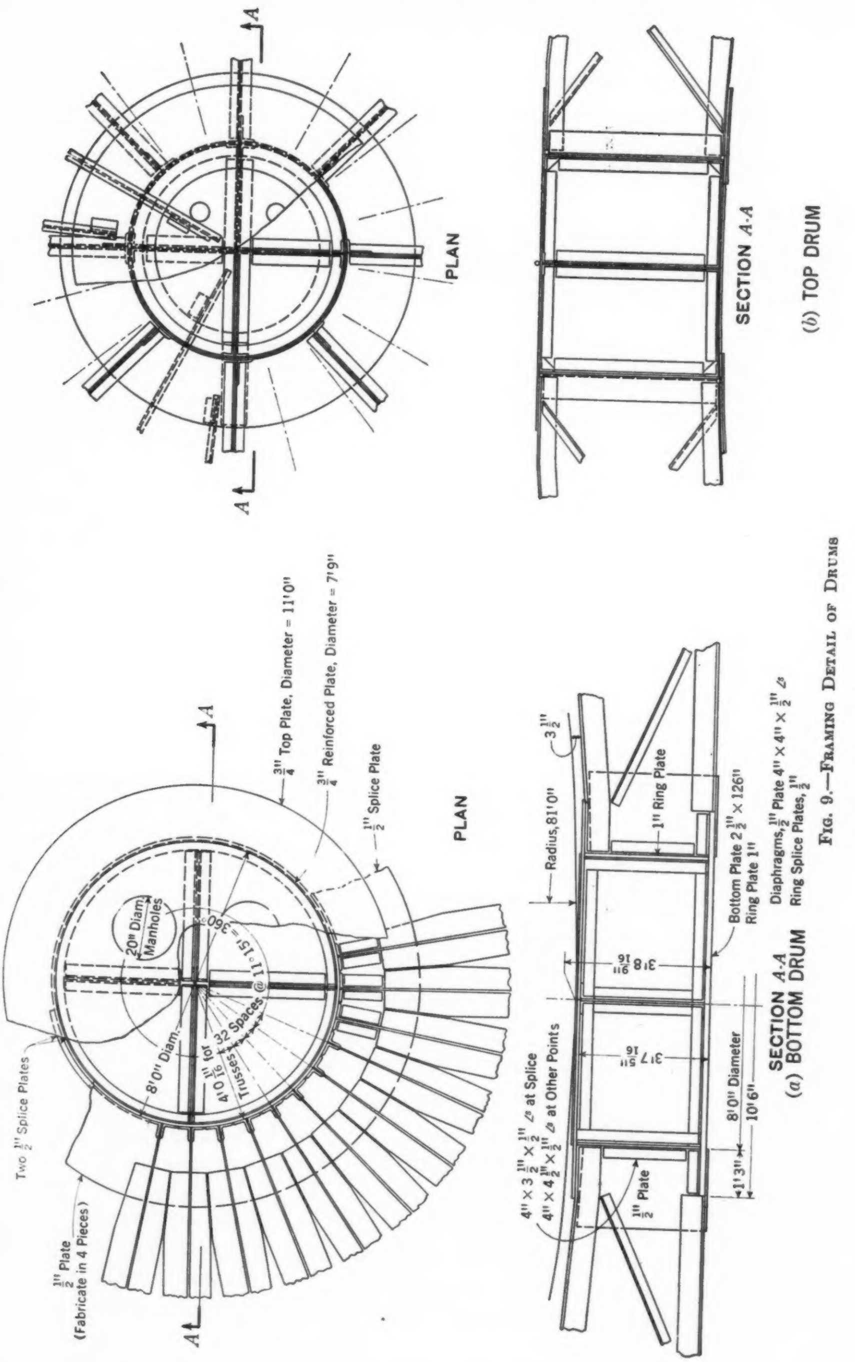



FIG. 9.—FRAMING DETAIL OF DRUMS

336 ft are box sections of plates and angles; those for the next 154 ft are 14-in. H-sections.

The unit connecting the Trylon and Perisphere is a combination through and deck bridge with its trusses 16 ft center to center. The electric stairways, one of which has the highest rise of any in the United States, run up through the Bridge, and the deck of the structure carries the walkway exit from the Perisphere.

## Perisphere

*Upper Hemisphere.*—For design purposes and facility of fabrication, the Perisphere was divided into two main parts—the upper hemisphere and the lower hemisphere.

The natural division of the Perisphere at its equatorial girt makes the upper part a full spherical framed dome. The analogy between a framed dome and a solid dome is quite readily realized, especially if the spacing of the meridians and girts is a minimum, and even more so when all the main members are securely tied together by an adequate bracing system and covering. This dome is of great size, but not of extreme thinness, and consequently the effect of unbalanced loads is of minor importance, except in the study of localized conditions. The unsymmetrical-loading state of stresses existing in a very stiff dome structure, such as this one, does not exceed those stresses due to full symmetrical loading. All surface loadings are considered to be concentrated at the panel points.

The general equation for meridional thrust is:

$$T = -\frac{W}{N}\sec\theta \qquad (1)$$

in which $W$ is the sum of all loads above any given level; $\theta$ represents the angle between the horizontal and the radius to this given level; and $N$ is the number, or equivalent number, of meridians at this level.

The direct stress in a girt is represented by the equation:

$$G = \left[\frac{W}{2\,\pi\,r}\sec^2\theta - (p\,x)\tan\theta\right]S \qquad (2)$$

in which $r$ is the radius; $x$ is the horizontal distance from the vertical axis to the point in question; $p$ represents the average unit weight of the structure at the girt level under consideration; and $S$ is the length of profile between midway points of adjacent panel points.

Fig. 8 presents some interesting data from the design of the upper dome. The distance of any girt above the equator is plotted vertically; and the angle that the great-circle plane within which it is embodied makes with the horizontal is designated. Plotted horizontally is the girt stress per foot of profile. Fig. 10(*a*) is the graphical plot for dead load of the upper hemisphere.

A concentrated zenith load on the structure is produced by the rotating scaffolding frame used for the purpose of placing the inner covering. Fig. 10(*b*) is the plot of the resulting girt stresses for this loading.

Fig. 10(c) represents three different sets of girt stresses for three differently assumed snow loadings. The loaded surface for all cases is that of a spherical segment 26 ft high. The uniform load curve is drawn on the assumption that the entire surface is subjected to a 25-lb-per-sq-ft loading. This assumption makes no allowance for the fact that, as the surfaces recede from the vertical axis of the Perisphere, they become steeper and hence cannot retain a 25-lb snow as do the more nearly horizontal surfaces. For the two other cases, the loading was assumed to have first a circular, and then a parabolic, variation from 25 lb per sq ft maximum to zero as a minimum. The latter two cases are perhaps closer to the actual load experienced by the structure than the first case.

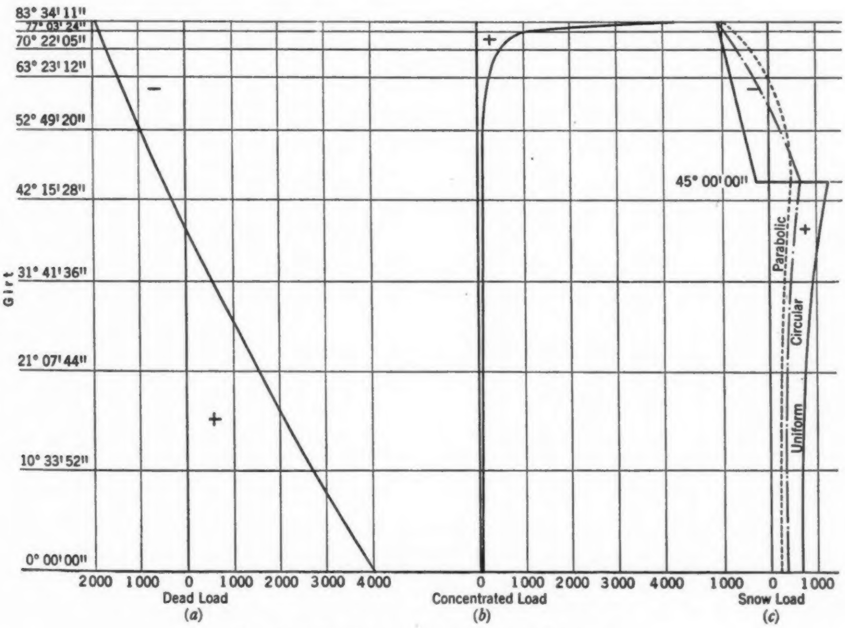



Fig. 10.—Girt Stress per Foot of Profile

All meridians and girts for the upper hemisphere, except the diagonal meridians and the girts near the top, are comprised of two-angle chord sections double laced with single angles. Since the chords of both the meridian and girt trusses are curved, a bending moment equal to the axial stress times the mid-ordinate of the curve between lacing intersections must be considered; and, since the girt chords act also as supporting members for the vertical purlin system, the unsymmetrical bending that occurs produces another set of stresses. In order to reduce the amount of unsymmetrical bending, a hanger was introduced from the intersection of the X-bracing. Since the bracing angles are straight from panel point to panel point, and the outer chord of the girt lies in a spherical surface, the hanger is skewed and produces a kick upon the trussed girt chord.

The center drum of the upper half of the Perisphere (see Fig. 9) consists of a flat $\frac{1}{2}$-in. plate 9 ft 3 in. in diameter acting as a splicing medium for the top of the eight meridians that attach to the drum as well as a top plate of the drum itself. The bottom plate of the drum is $\frac{1}{2}$ in. thick and 5 ft 8 in. in diameter. A $\frac{1}{2}$-in. ring splice plate is used for fixing the bottom chords of the eight meridians to the drum. The vertical ring of the drum is comprised of a $\frac{3}{4}$-in. plate 4 ft 9 in. deep and two angles 4 in. by 4 in. by $\frac{1}{2}$ in. bent to conform to a diameter of 5 ft 10 in. The meridian trusses attach to this vertical ring plate by means of 8-in. by 4-in. by $\frac{1}{2}$-in. angles. This detail, further aided by the two vertical cross diaphragms, made up of $\frac{1}{2}$-in. plates and 4-in. by 4-in. by $\frac{1}{2}$-in. angles, forms a rigid unit capable of transmitting the shearing forces.

The principal meridian stresses for the upper hemisphere are shown in Table 3.

TABLE 3.—PRINCIPAL MERIDIAN STRESSES FOR THE UPPER HEMISPHERE

| Member[a] | Dead Load | | Snow Load | | Wind Load | | Section | |
|---|---|---|---|---|---|---|---|---|
| | Direct, in kips | Bending stress, in kip-in. | Direct, in kips | Bending stress, in kip-in. | Direct, in kips | Bending stress, in kip-in. | Angles | Double lacing |
| 27-26 | −25 | 13 | − 5 | 2 | −25 | 12 | Four, $6 \times 3\frac{1}{2} \times \frac{5}{16}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 26-25 | −17 | 4 | − 8 | 4 | −25 | 7 | Four, $6 \times 3\frac{1}{2} \times \frac{5}{16}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 25-24 | −25 | 6 | −12 | 3 | −25 | 6 | Four, $6 \times 3\frac{1}{2} \times \frac{5}{16}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 24-23 | −17 | 4 | − 9 | 4 | −25 | 10 | Four, $6 \times 3\frac{1}{2} \times \frac{5}{16}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 23-22 | −24 | 7 | −12 | 4 | −25 | 6 | Four, $6 \times 4 \times \frac{3}{8}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 22-21 | −30 | 10 | −16 | 5 | −25 | 8 | Four, $6 \times 4 \times \frac{3}{8}$ | $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ |
| 21-20 | −37 | 12 | −15 | 5 | −25 | 8 | Four, $6 \times 4 \times \frac{3}{8}$ | $3 \times 3 \times \frac{5}{16}$ |
| 20-19 | −46 | 14 | −13 | 4 | −25 | 8 | Four, $6 \times 4 \times \frac{3}{8}$ | $3 \times 3 \times \frac{5}{16}$ |
| 19-18 | −53 | 36 | −12 | 8 | −25 | 17 | Four, $6 \times 4 \times \frac{3}{8}$ | $3 \times 3 \times \frac{5}{16}$ |
| 18-17 | −64 | 43 | −12 | 8 | −25 | 17 | Four, $6 \times 4 \times \frac{3}{8}$ | $3 \times 3 \times \frac{5}{16}$ |

[a] See Fig. 12.

*Girts.*—Table 4 indicates the stresses occurring in the girt trusses of the Perisphere. As listed, there are fifteen girt trusses in all, nine and five in the upper and lower halves, respectively, and the equatorial girt which is common to both halves of the structure. The tabulated stresses listed for unsymmetrical bending are those for the outer chords of the girts only. In girts 17 to 7, inclusive, the stresses resulting from snow load are included in the values listed under the dead-load columns. As will be noted, for main action, the girt truss at panel points 13 is completely idle and the one at panel point 7 partly so. In Fig. 8 is shown the one opening in the sphere, approximately 34 ft square, into which projects, without imposing any load on the Perisphere, the escalator and walkway unit—the Bridge. Being two panels in height, this aperture breaks the continuity of girt truss 13. As a result of this interruption, the lower half of the Perisphere was structurally designed as if girt 13 had been omitted; at the same time the girt was necessary at this location to carry the purlin systems, platforms, and coverings. The requirement that the structure be allowed to deform as designed, without any restraining action on the part of this discontinuous girt, was accomplished by making its panel-point connections with $\frac{7}{8}$-in. bolts in $1\frac{1}{16}$-in. holes, the nuts on the bolts being drawn up

lightly and the threads checked. In adhering to the analysis of the lower hemisphere, and in order to control more closely the $B_5$ point, there must be no force exerted at the inner panel point of girt truss 7. Hence, these panel points were treated in the same manner as those of girt truss 13. The outer panel point of girt 7 has little or negligible dead-load stress, and as a result no control over the $B_5$ point; consequently it was riveted tight.

Girt 9 has an unsymmetrical inner flange whose section is comprised of an 8-in. by 6-in. angle and a 6-in. by 6-in. angle. The area provided by this inner flange is in excess of that required for stress, but was made necessary in order to accommodate the proper seating, connections, and framing for the bases of the moving platform columns.

TABLE 4.—STRESSES IN PERISPHERE GIRT TRUSSES

| Girt member at panel point: | DEAD LOAD | | | SNOW LOAD | | | WIND LOAD | | SECTION | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Direct, in kips[a] | Bending Moments, in Kip[a]-In. | | Direct, in kips[a] | Bending Moments, in Kip[a]-In. | | Direct, in kips[a] | Bending moment, in kip[a]-in. | Angles | Double lacing |
| | | Direct | Unsymmetrical | | Direct | Unsymmetrical | | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) |
| 26 | − 20 | 9 | 28 | −11 | 5 | 23 | − 7 | 31 | Two, 6×4×3/8 | 3×3×5/16 |
| 25 | − 19 | 10 | 28 | −11 | 6 | 23 | − 8 | 33 | Two, 6×4×3/8 | 3×3×5/16 |
| 24 | − 17 | 6 | 14 | −10 | 4 | 14 | −11 | 17 | Two, 6×6×3/8 | 3×3×5/16 |
| 23 | − 20 | 11 | 31 | −11 | 6 | 26 | −11 | 35 | Two, 6×6×3/8 | 3×3×5/16 |
| 22 | − 17 | 6 | 11 | − 9 | 3 | 10 | −15 | 13 | Four, 6×3½×3/8 | 3×3×5/16 |
| 21 | − 6 | 2 | 23 | +20 | 7 | 22 | −20 | 20 | Four, 6×3½×3/8 | 3×3×5/16 |
| 20 | + 9 | 4 | 37 | +15 | 6 | .... | −20 | 28 | Four, 6×4×3/8 | 3×3×5/16 |
| 19 | + 26 | 17 | 37 | +14 | 8 | .... | −22 | 24 | Four, 6×4×3/8 | 3×3×5/16 |
| 18 | + 44 | 32 | 30 | +12 | 8 | .... | −25 | 34 | Four, 6×4×7/16 | 3×3×5/16 |
| 17 | +164 | 46 | 30 | .... | .... | .... | −20 | 60 | Four, 6×6×½ | 3×3×5/16 |
| 15 | +190 | 94 | 41 | .... | .... | .... | +42 | 80 | Four, 6×6×9/16 | 3½×3×5/16 |
| 13 | .... | .... | 26 | .... | .... | .... | .... | .... | Four, 6×6×½ | 3½×3×5/16 |
| 11 | +328 | 112 | 21 | .... | .... | .... | +35 | +41 | Two, 8×8×¾<br>Two, 8×8×7/8 | 4×3×5/16 |
| 9 | +109 | 34 | 16 | | | | − 8 | 32 | One, 8×6×9/16<br>One, 6×6×7/16<br>Two, 6×4×9/16 | 4×3×5/16 |
| 7 | .... | .... | 10 | | | | +21 | 42 | Four, 6×4×7/16 | 4×3×5/16 |

[a] 1 kip ("kilo-pound") = 1,000 lb

*Lower Hemisphere.*—The lower hemisphere of the Perisphere was designed by a process of trial and error, involving a series of converging adjustments in sectional areas, unit stresses, deformations, and total stresses. Each meridian truss below the equator was assumed to be a cantilever truss supported vertically on the ring girder and fixed horizontally at the vertical axis of the sphere; loaded at its upper end with the vertical loads from the upper hemisphere and at its various panel points with the vertical loads of the steelwork, inner and outer covering, circular platforms, equipment, and live loads; and restrained against outward movement by horizontal pulls from the various girts. The vertical loads were known, or could be assumed with sufficient accuracy. As a first approximation, a set of horizontal forces from the girts was assumed that was just large enough to balance the moment of the vertical forces. The stresses in the various members of the cantilever truss were then computed, sectional areas assumed, and the outward deflections of the various panel points of the truss

computed by means of a Williot diagram. From the outward movement at the level of each girt, the resulting unit stresses in the girt could be computed. Since the first set of girt pulls had been assumed arbitrarily, the resultant horizontal displacements and girt stresses were not satisfactory, the cantilever trusses being bent considerably out of shape, and some of the girts showing very high tension and others considerable compression. After an examination of the results, a new set of horizontal pulls was assumed; the stresses, sections, deformations, and deflections of the cantilever truss recomputed; and the

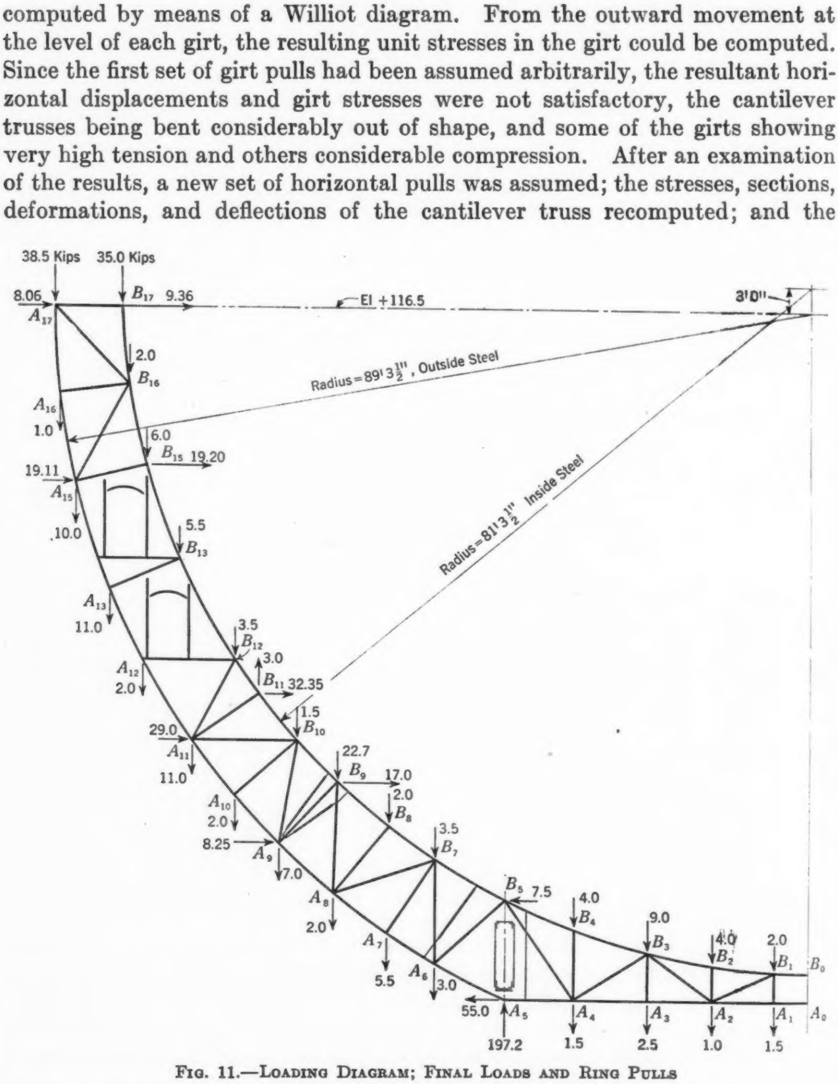



FIG. 11.—LOADING DIAGRAM; FINAL LOADS AND RING PULLS

resultant girt unit stresses again computed. After a number of successive trials, it was found possible to arrive at a series of horizontal pulls that would give reasonably uniform unit stresses of the desired intensity in a set of girts of moderate cross-sectional areas. It was found further that if any attempt were made to vary appreciably the various horizontal pulls thus determined, the cantilever truss would be forced considerably out of shape, and a balance between girt pulls, girt unit stresses, and girt sections could not be reached. In other words, it was found that the girt stresses could not differ appreciably

from those arrived at. A change of as little as 100 lb in the horizontal pull applied by a given girt would throw the cantilever truss appreciably out of shape. It was also found that the cantilever trusses would be very rigid, since the girts would permit only very small transverse movements.

The action of the steel framework of the Perisphere under the dead, live, and snow loads is one in which the meridians, in compression, tend to bow out; this tendency in turn brings the girts into play as tension rings. One can have a very clear mental picture of the entire structural action by pressing uniformly on top of a rubber ball set in a ring collar about one third the diameter of the ball. In reacting to these dead, live, and snow loads the Perisphere will deform from the outline of a true sphere. When it is subjected to final loads and ring pulls as indicated in Fig. 11, the Perisphere will deform so as to allow a point on the equator to move radially 0.25 in. and vertically 0.5 in. The principal meridian stresses for the lower hemispheres are shown in Table 5.

TABLE 5.—PRINCIPAL MERIDIAN STRESSES FOR THE LOWER HEMISPHERE

| (a) OUTSIDE STEEL | | | | | | | (b) INSIDE STEEL | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Member[a] | Direct load,[b] in kips | Bending stress,[b] in kip-in. | Section | | | | Member[a] | Direct load,[b] in kips | Bending stress,[b] in kip-in. | Typical section (two angles) |
| | | | Typical | | Entrance | | | | | |
| | | | Two angles | One plate | Two angles | One plate | | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (1) | (2) | (3) | (4) |
| A17-A15 | −27 | 50 | 6×6×7/16 | .... | .... | .... | B17-B16 | −38 | 63 | 6×6×7/16 |
| A15-A11 | −25 | 56 | 8×6×9/16 | .... | .... | .... | B16-B15 | −59 | 100 | 6×6×9/16 |
| A11-A9 | −53 | 56 | 6×6×7/16 | 7×5/8 | 6×6×1/2 | 7×5/8 | B15-B12 | −103 | 257 | 8×6×9/16 |
| A9 -A8 | −65 | 86 | 6×6×7/16 | 7×5/8 | 6×6×1/2 | 7×5/8 | B12-B10 | −89 | 46 | 8×6×9/16 |
| A8 -A7 | −113 | 97 | 8×6×9/16 | 7×5/8 | 8×6×11/16 | 7×5/8 | B10-B9 | −122 | 114 | 8×6×3/4 |
| A7 -A6 | −117 | 97 | 8×6×9/16 | 7×3/4 | 8×6×11/16 | 7×3/4 | B9 -B7 | −127 | 145 | 8×6×3/4 |
| A6 -A5 | −209 | .... | 8×8×13/16 | .... | .... | .... | B7 -B5 | −51 | 93 | 8×8×13/16 |
| A5 -A4 | −132 | .... | 8×8×3/4 | 6½×3/4 | 8×8×7/8[c] | .... | B5 -B3 | +40 | 62 | 8×8×3/4 |
| A4 -A2 | −101 | .... | 8×8×3/4 | .... | .... | .... | B3 -B1 | −5 | .... | 8×8×3/4 |
| A2 -A1 | −70 | .... | 8×8×3/4 | .... | .... | .... | .... | .... | .... | .... |

[a] See Fig. 12. [b] Dead load plus live load. [c] At columns.

The emergency exit fire passageways contained between the outer and inner shells of the Perisphere at the 75.5-ft and 87.5-ft levels necessitated the plate section of meridian truss between the 12th and 15th panels.

In Table 5(*a*) (lower hemisphere) it will be noted that the sections of the outer meridian chords between panel points 11 and 6 are not the usual two-angle section, but have plates 7 in. deep placed between the angles and extending from gusset plate to gusset plate, but not developed into the joints. This type of member affords ample chord section for direct stress in the panel point region as well as for the increased stress due to bending at the mid-panel point. Table 5 (lower hemisphere) indicates heavier chord members below panel point 9 at the entrance side of the Perisphere, which was done to accommodate the increase in stresses due to the extra dead and live loads concentrated at the junction of the Perisphere and Bridge. The net unbalanced steel load of 50,000 lb at the entrance to the Perisphere is partly counteracted by placing,

advantageously, some of the heavier electrical equipment in the four bays diametrically opposite the entrance. Another aid in balancing the extra load at the entrance is the placing of a concrete carrying slab for the electrical apparatus as compared with the lighter composition floor on the entrance platforms.

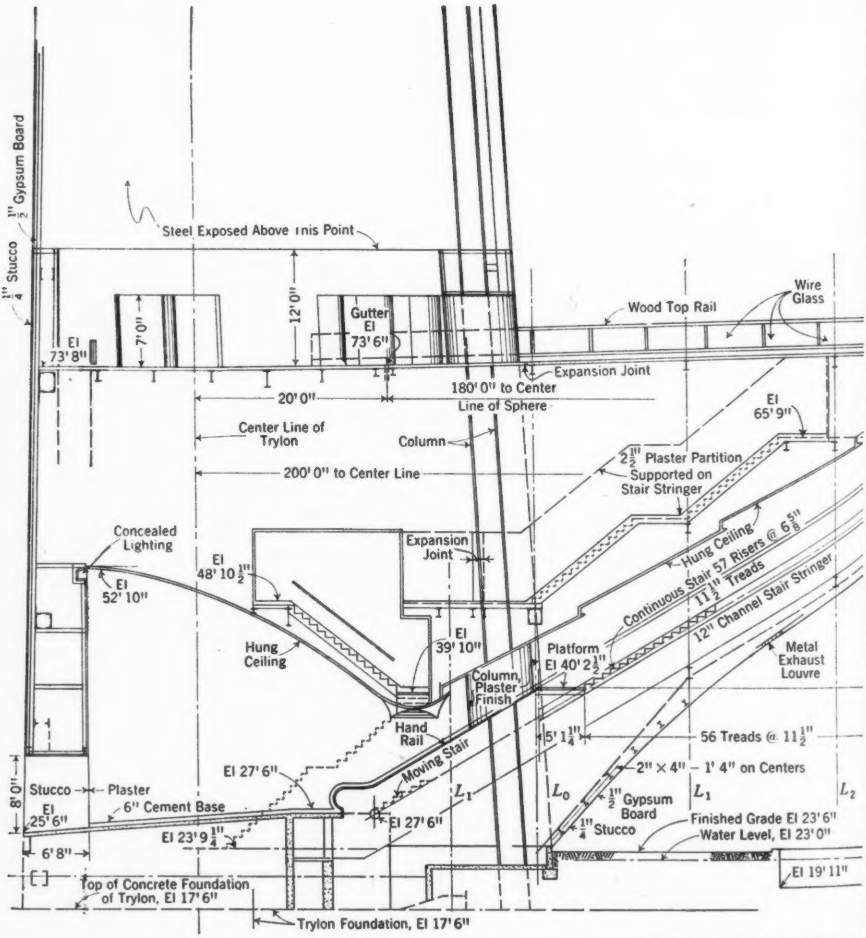
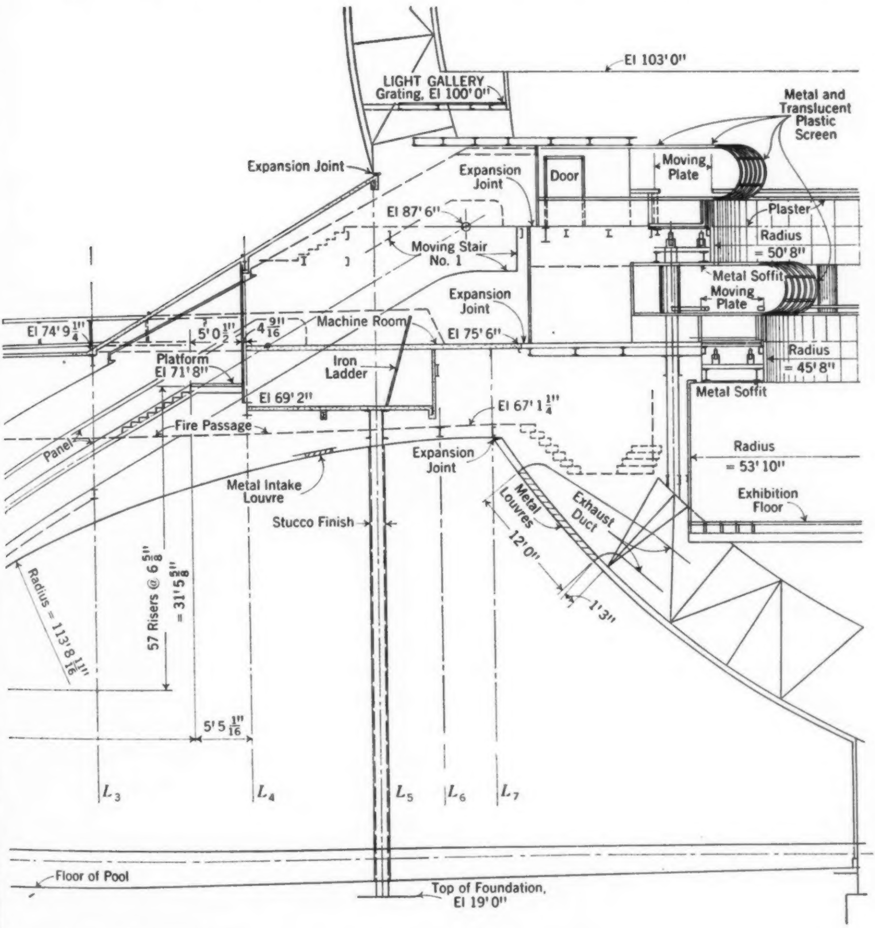



FIG. 12.—GENERAL SECTION

In the final analysis, due to the nature of the structure (that is, its geometrical arrangement), the Perisphere framework was found to be extremely sensitive to even small changes in the assumed distribution of stresses in the various parts. This supersensitive characteristic of the structure is a measure of design safety. Since the entire framework strives to distribute quickly all types of loadings, it is of prime importance that very careful study be accorded the connections and framing of all members. Inasmuch as the structure was

designed to conform to particular deformations, resulting in specified sets of stresses, care was taken to avoid, as far as it was physically possible, any interference by members or their connections with the desired deformations of the structure.

The ring girder as a member receives its stress from bending due to vertical




THROUGH THEME CENTER

loads and ring shortening caused by meridional thrusts. The chords of the meridian trusses pass above and below the flanges of the ring girder. This arrangement facilitated fabrication and erection but complicated the connection for transferring meridional thrusts into the ring girder; and a further complication which makes this particular connection of the utmost importance is the abrupt change in the path of the outer chord stresses caused by the outer meridional chords turning horizontally at this point.

In the outer cross bracing system, just outside the ring girder, there is a two-angle wind strut connected to the bottom cover plates of the ring girder by $\frac{5}{8}$-in. bent plates. By means of this device the wind shear is transferred into the bottom girder flange. In order that this strut, which is in reality an auxiliary ring flange, shall be reserved for wind action only and shall not participate in dead-load deformations, it should, theoretically, have been left unriveted until the dead load was entirely placed. This procedure, of course, was a practical impossibility; hence the desired action was accomplished in the most part by leaving the riveting of these struts to the meridional gusset plates until after all other rivets in the steel superstructure had been driven—the steel superstructure being 50% of the entire dead load of the structure.

*Moving Platforms.*—The design of the lower half of the Perisphere includes two directionally opposed moving platforms. It is from these moving platforms traveling at the rate of 60 ft per min that spectators have an unobstructed view of the entire interior of the Perisphere.

These platforms, as shown in Fig. 12, are fed by two moving stairways, an upper and lower level within the Bridge unit, at the rate of 90 ft per min. These 2-ft wide moving stairways, having an angle of rise of 30°, are driven electrically. As shown in Fig. 12, the two moving platforms are on different levels, one 12 ft above the other. The platforms are 6 ft wide, and have a stationary rear railing and a moving floor and front railing propelled at the quarter points by sets of driving wheels and motors. The entire journey, from the time of entering to the time of leaving the Perisphere, requires $5\frac{1}{2}$ min.

The moving platforms and driving mechanisms are carried on I-beams spanning between the sixteen hammer-head columns. The sixteen columns, commencing with the center meridian, rest on alternate meridian trusses at inside panel points $B_9$. The columns, which were analyzed as part of a continuous frame, are braced circumferentially by a strut at the 75.5-ft level, whose section is made up of two 18-in. channels, a 14-in. cover plate, and $2\frac{1}{2}$-in. by $\frac{3}{8}$-in. double lacing. This strut reduced by half the stresses due to bending in the columns, and also afforded rigidity against any vibration caused by the motion of the platforms. Still further rigidity is obtained by means of the two panels of X-bracing between the columns at the entrance to the Perisphere.

*Distributing Trusses.*—As has been previously stated, there are only sixteen columns supporting the two moving platforms, and of these the three at the entrance to the Perisphere carry a higher live load, due to the massing of people in this vicinity, and a heavier dead load because of the extensive framing and partitioning necessary in order to provide the required floor areas (see Fig. 13).

These sixteen columns find their seats on sixteen meridian trusses at the inside panel points $B_9$. Since there are thirty-two meridians that must be made to share alike in carrying this platform loading to produce uniform deformation of the framework, a truss was designed to frame between meridians at the load-application points. The inside chord of the girt at panel points $B_9$ serves also as the top chord for the distributing truss.

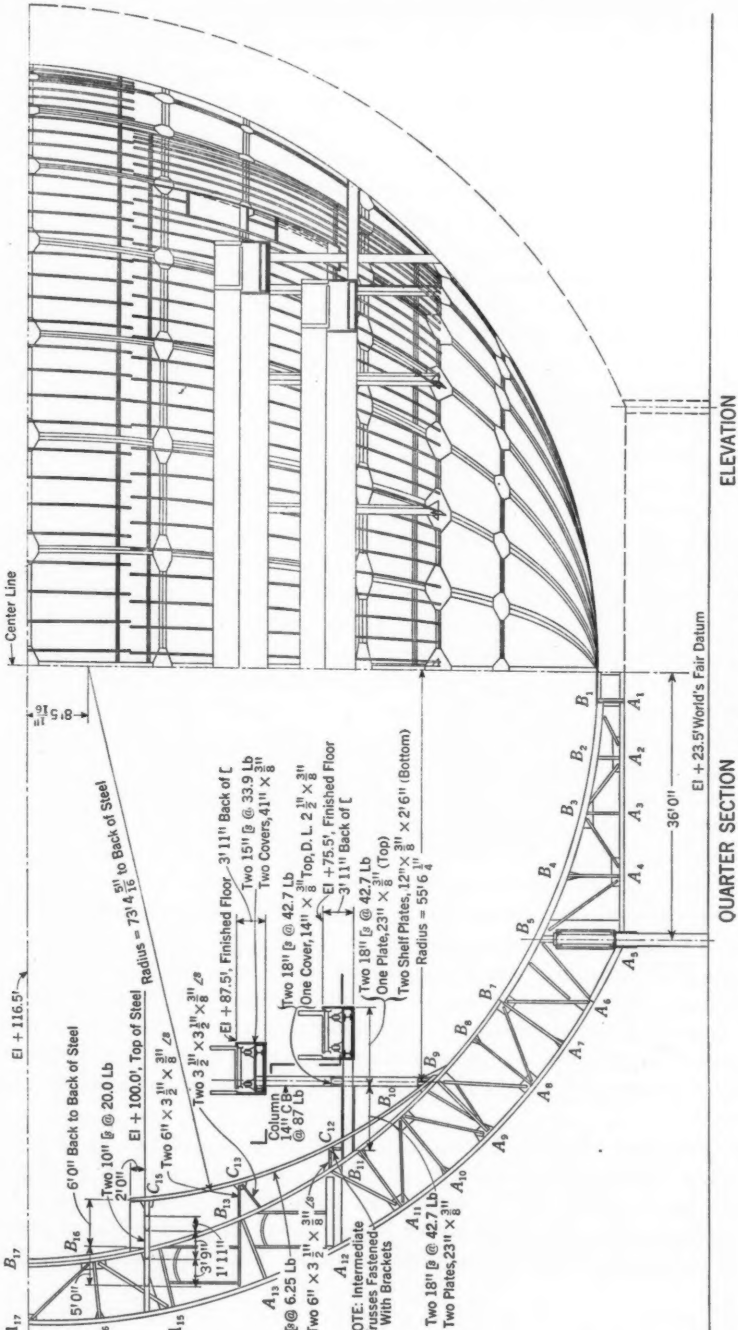



Fig. 13.—Steel Framing, Lower Hemisphere

The diagonals $A_8B_9$ of the meridian trusses form the verticals in the distributing truss. The verticals in the distributing truss are, at the column load points, two angles 6 in. by 6 in. by $\frac{3}{8}$ in., whereas at the non-load points they are of the lighter section, two angles 6 in. by $3\frac{1}{2}$ in. by $\frac{3}{8}$ in. The $A_8B_9$ diagonals of the distributing truss are made up of two angles 6 in. by 4 in. by $\frac{7}{16}$ in.

At the entrance to the Perisphere the distributing truss, in order to provide for the greater and unbalanced loading, has stronger sections and a bottom chord, as well as a top chord, for eight panels. All diagonal sections remain the same as in the other parts of the distributing truss. The bottom-chord section consists of two panels of two angles 4 in. by 3 in. by $\frac{1}{2}$ in. and two panels of two angles 4 in. by 3 in. by $\frac{3}{8}$ in., symmetrical about the center meridian. The difference in the verticals occurs at the center meridian, the first panel point, and the first column load point out from this center meridian; these are two angles 8 in. by 6 in. by $\frac{3}{4}$ in., two angles 6 in. by 6 in. by $\frac{1}{2}$ in., and two angles 8 in. by 6 in. by $\frac{1}{2}$ in., respectively.

*Ring Girder.*—As previously explained, the inner and outer chords of the meridian trusses pass above and below the ring girder, respectively. The details of the connections of these truss chords to the top and bottom of the ring girder are such that they form a rigid unit. The heavy compression stresses occurring in the outer chords of the meridian trusses deform the bottom flange of the ring girder inward. An attempt was made in the final analysis to make the stresses occurring in the inner chords of the meridian trusses such that the horizontal movement of the panel points located vertically above the top flange of the ring girder would equal zero. This objective was not quite attained, because these panel points move outward a very small amount.

The combined motion of panel points $A_5$ and panel points $B_5$, pushing the bottom flange of the ring girder in and pulling the top flange of the ring girder out, makes this member take a cone-like shape. This type of distortion caused by meridian thrusts necessitates compression in the bottom flange and tension in the top flange of the ring girder, in magnitudes directly proportional to the amount of the deformations that occur.

The ring girder is a continuous box-section girder 72 ft in diameter, weighing 139 tons. This unit with its bottom flange 12 ft 6 in. above the ground surface is the medium for transferring the loads and forces from the Perisphere framework into the columns and thence to the foundations. Fig. 14 shows the ring girder completely assembled without columns, but with the bottom center drum and the first division of meridian trusses, at the fabricating plant. The girder was fabricated in eight identical pieces, with the bottom flanges milled to bear.

All thirty-two meridian trusses frame into the ring girder in passing on to butt at the bottom center drum detail. The girder as a member receives its stresses from bending due to vertical loads, and ring shortening caused by meridional thrusts. The ring girder is supported on eight columns and receives load from the thirty-two meridian trusses. Since there is meridian truss

at each column, it follows that a section of girder between two adjacent columns is loaded at its midpoint and its two quarter points by the other meridian trusses.

In the final analysis of the 72-ft diameter girder for direct stress as a ring, panel points $B_5$ and $A_5$ were found to move out 0.015 in. and to move in 0.058 in., respectively.

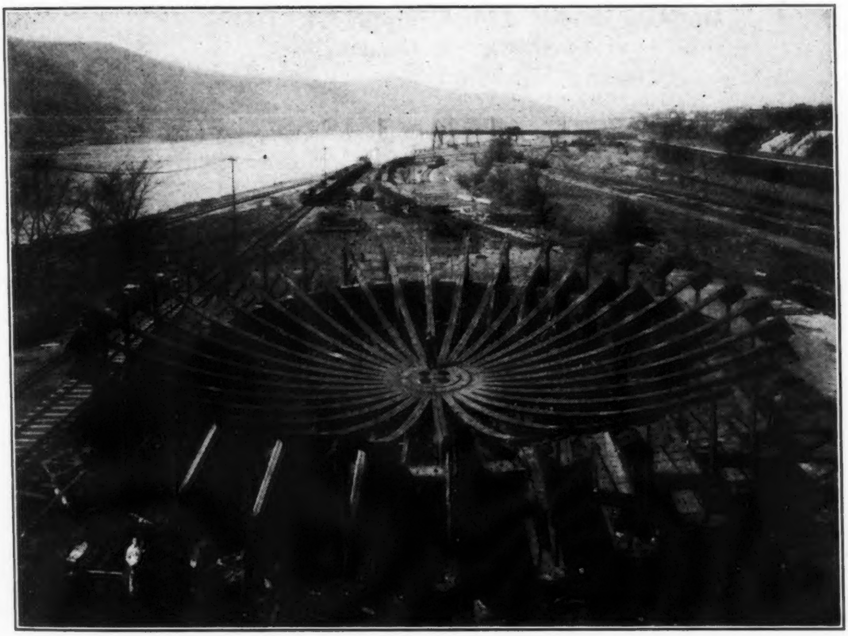


FIG. 14.—TEST ASSEMBLY OF RING GIRDER AND BOTTOM DRUM AT THE FABRICATING PLANT

The assumption was made that these deformations would create ring stresses in the circular girder. The flanges of the ring girder were then sectioned so as to produce the unit stresses accompanying the foregoing deformations—tension in the top flange and compression in the bottom flange. Since the ring-girder flanges are necessarily connected, one to the other, by the two $\frac{7}{16}$-in. webs, some participation in the ring action must be taken by these webs. After due consideration it was assumed that 45 in. of the webs played a part in the action, the maximum web participation being at the bottom flange and decreasing as a straight-line variation to zero at a point 45 in. up from the bottom flange.

The following data indicate the ring-girder sections used and the stresses from balanced loads:

| Item | Description | |
|---|---|---|
| | Section: | |
| 1 | Two web plates | 90 in. by $\frac{7}{16}$ in. |
| 2 | Four angles | 8 in. by 6 in. by $\frac{3}{4}$ in. |
| 3 | One top cover plate | 23 in. by $\frac{5}{8}$ in. |
| 4 | Two bottom cover plates | 23 in. by $1\frac{3}{16}$ in. |
| 5 | Maximum dead-load moment, in pound-feet | 2,133,000 |
| 6 | Maximum dead-load shear, in pounds | 362,000 |
| | Dead-Load Unit Stress at Column, in Pounds per Square Inch: | |
| 7 | Vertical bending | 2,940 |
| 8 | Ring shortening (from meridional thrust) | 3,490 |
| 9 | Total | 6,430 |

*Bottom Drum.*—After passing the ring girder the meridian trusses converge on a center drum detail. All bottom chords of the thirty-two meridian trusses bear with milled surfaces against the bottom plate of the center drum which is 10.5 ft in diameter and $2\frac{1}{2}$ in. thick, as shown in Fig. 9. A 1-in. ring plate stiffened by plate and angle diaphragms, a top plate 11 ft in diameter and $\frac{3}{4}$ in. thick, and a top reinforcing plate $\frac{3}{4}$ in. thick make up the remainder of the drum. All meridian trusses are rigidly framed into the top, bottom, and ring plates of the drum detail.

*Air Conditioning.*—The complete air conditioning system provided for the Perisphere requires a fully equipped fan and compressor room. This room and equipment were placed within the ring girder and supported on the inner flanges of the meridian trusses. The reinforced concrete platform forming the floor of the room is placed on a series of circular, concentric, reinforced concrete beams resting on the panel points of the meridian flanges and in this manner distributing the superimposed load uniformly to the meridians. This load, placed within the ring girder, actually decreases somewhat the meridian stresses resulting from the action of the framework. The accompanying ductwork and details of the system are contained within the space provided between the outer and inner shells, and the necessary louvers and vents are provided near the top of the Perisphere.

*Columns.*—The make-up of the eight main columns, each weighing 9 tons, supporting the Perisphere is as shown in Fig. 15. Because of the framing of these columns into the ring girder and meridian trusses, they are fixed at the top. At the bases the columns are fixed, by anchor bolts 3 in. in diameter, about the radial axis 2–2; but in order to reduce temperature stresses the columns are hinged about the tangential axis, 1–1.

The hinging effect was accomplished, as will be noted in Fig. 15, by tapering the 5-in. base plate and providing the 8-in. by 8-in. by $\frac{1}{2}$-in. breathing angles.

*Wind Shear on Columns (See Fig. 16).*—The distribution of the wind shear among the various columns was determined in the following manner: All columns are alike. At the bottom they are pin-ended about axis 1–1 and fixed-

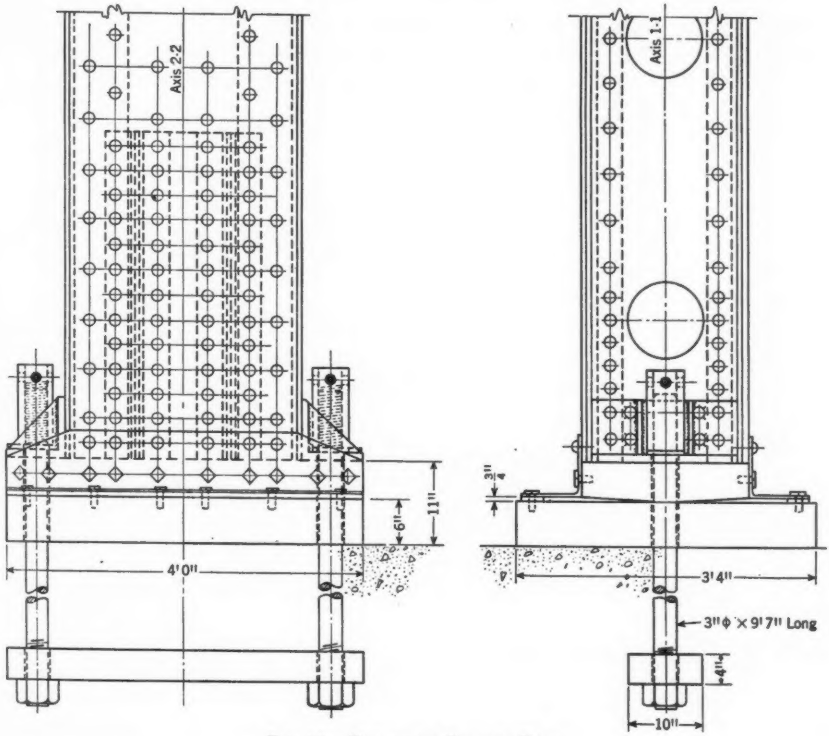



FIG. 15.—DETAIL OF COLUMN BASE

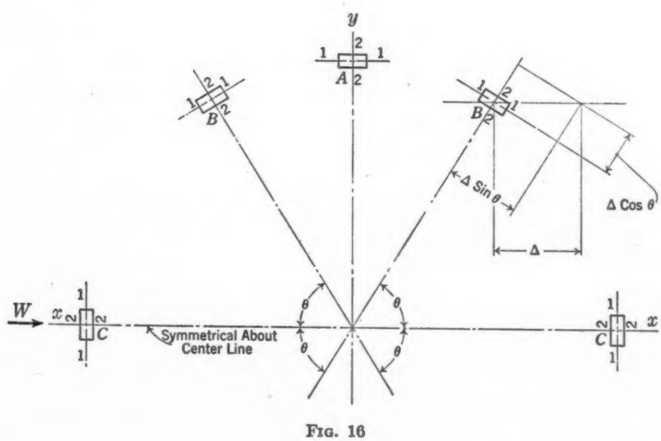



FIG. 16

ended about axis 2–2. Let $I_1$ and $I_2$ be the moments of inertia about axes 1–1 and 2–2, respectively; and let $L$ = length of column and $W$ = force. Now subject the sphere to a horizontal movement $\Delta$ along the $x$-axis. Furthermore, let $V$ = the resistance to a movement $\Delta$, along the $x$-axis, developed by a given column about axis 1–1 or 2–2. For example, $V_{A2}$ = resistance developed by column $A$ about axis 2–2. Then:

$$2\,V_{A2} + 2\,V_{C1} + 4\,V_{B1}\cos\theta + 4\,V_{B2}\sin\theta = W \qquad (3)$$

and

$$\frac{V_{A2}\,L^3}{12\,E\,I_2} = \Delta = \frac{V_{B1}\,L^3}{3\,E\,I_1\cos\theta} = \frac{V_{B2}\,L^3}{12\,E\,I_2\sin\theta} = \frac{V_{C1}\,L^3}{3\,E\,I_1} \qquad (4)$$

Hence in terms of $V_{A2}$:

$$V_{B1} = \frac{I_1}{4\,I_2}\cos\theta\,V_{A2} \qquad (5a)$$

$$V_{B2} = V_{A2}\sin\theta \qquad (5b)$$

and

$$V_{C1} = \frac{I_1}{4\,I_2}\,V_{A2} \qquad (5c)$$

Substituting the values of $V_{B1}$, $V_{B2}$, and $V_{C1}$ in Eq. 3,

$$V_{A2}\left(2 + \frac{I_1}{2\,I_2} + \frac{I_1}{I_2}\cos^2\theta + 4\sin^2\theta\right) = W \qquad (6)$$

For $\theta = 45°$, and letting $\frac{I_1}{I_2} = K$:

$$V_{A2} = \frac{W}{4 + K}; \qquad M_{A2} = V_{A2}\frac{L}{2}$$

$$V_{B1} = \frac{0.707}{4}K\left(\frac{W}{4 + K}\right); \qquad M_{B1} = V_{B1}\,L$$

$$V_{B2} = 0.707\left(\frac{W}{4 + K}\right); \qquad M_{B2} = V_{B2}\frac{L}{2}$$

and

$$V_{C1} = \frac{K}{4}\left(\frac{W}{4 + K}\right); \qquad M_{C1} = V_{C1}\,L$$

*Temperature Stresses in Columns.*—There is temperature bending about axis 1–1 only—that is, temperature bending occurs only in a radial direction. Then the temperature bending moment is

$$M_t = \frac{3\,E\,I_1\,\Delta_t}{L^2} \qquad (7)$$

in which $E$ = modulus of elasticity; $I_1$ = moment of inertia about axis 1–1; $\Delta_t$ = deflection due to temperature; and $L$ = length of column. Substituting the values for a temperature change of 40° F: $M_t = 0.282\,I_1$ kip-in. $= \frac{0.282 \times 13{,}561}{12} = 319$ kip-ft.

*Column Section.*—The gross area, in square inches, is:

| | |
|---|---|
| 4 plates 30 by $\frac{11}{16}$ | 82.52 |
| 4 angles 8 by 4 by $\frac{3}{4}$ | 33.76 |
| 2 plates 20 by $\frac{5}{8}$ | 12.50 |
| Gross area | 128.78 |

The section moduli are: For axis 2–2, 917; and for axis 1–1, 1,164. The unit stresses are as follows:

| | |
|---|---|
| Direct stress | $\frac{1,022,000}{128.78} = 8,000$ |
| Bending stress | $\left\{\begin{array}{l}\frac{7,872,000}{917} = 8,600 \\ \frac{1,915,000}{1,164} = 1,600\end{array}\right.$ |
| Dead + Live + Snow + Wind, in pounds per square inch | 18,200 |
| Temperature | 3,300 |
| Dead + Live + Snow + Wind + Temperature | 21,500 |

## Trylon

Because the wind forces governed the design of the Trylon, the columns as well as the anchorages are unusually heavy. The wisp-like appearance of the completed tower is belied by the fact that each tower-leg is anchored by means of a very heavy welded-riveted base and fourteen $2\frac{7}{8}$-in. diameter bolts each 12 ft in length. The base and the anchor bolts of each column are embedded in a three-way reinforced mass of concrete. Special precautions were taken in designing each tower leg for the 855-ton uplift. As is noted in the data table, it was necessary to change a large portion of the covering on the Trylon in 1940. The new type of covering weighed only a little more than one third of the type it replaced. The decrease in the weight of the covering increased the maximum uplift load to 920 tons on a column.

*First Column Section.*—The gross area, in square inches, is:

| | |
|---|---|
| 1 cover plate 22 by $1\frac{1}{8}$ | 24.75 |
| 2 angles 8 by 6 by 1 | 26.00 |
| 2 web plates 32 by $1\frac{1}{2}$ | 96.00 |
| 2 angles 8 by 8 by 1 | 30.00 |
| 2 plates 24 by 1 | 48.00 |
| 1 cover plate 37 by $1\frac{1}{8}$ (23 in. effective) | 25.88 |
| Gross area | 250.63 |

$$\frac{l}{r} = 32; \qquad \text{allowable } f_c = 14,740 \text{ lb per sq in.}$$

The unit stresses, in pounds per square inch, are as follows:

| | | |
|---|---|---|
| Dead load......... | $\frac{1,100,000}{250.63} =$ | 4,400 |
| Wind load......... | $\frac{2,580,000}{250.63} =$ | 10,300 |
| Dead + wind................. | | 14,700 |

## Coverings

The exterior covering of the Perisphere consists of 2-in. by 4-in. wood nailers, treated with a fire-retardant medium and screw-fastened to the vertical steel purlin system; two thicknesses of ½-in. gypsum wallboard, the first layer

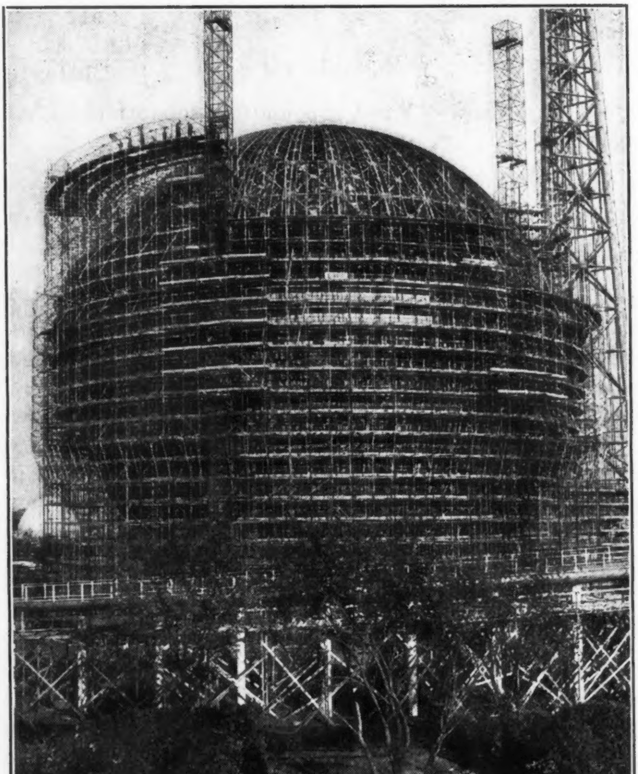


Fig. 17.—View of Demountable Scaffold System

being damp-proofed with a full covering of asphalt emulsion, were applied vertically and nailed to the horizontal timber furring. Then three layers of a magnesite compound, reinforced with two layers of jute fabric, were applied to the entire surface of the Perisphere and painted white.

The interior shell or lining of the Perisphere consists of an acoustical tile, upon which pictures are projected. The tile is nailed to 2-in. by 3-in. fire-proofed horizontal furring, 27 in. on centers, attached to the inner steel purlins by means of heavy wire clips. On the area of tile extending from the offset in the inner shell below the equator, to a point approximately 30° from the zenith, the tile has a backing in alternate panels, in both directions, of a $\frac{3}{8}$-in. plasterboard and a 1-in. sound-absorbing board.

The exterior covering applied to the steel purlins of the Trylon and Bridge was of the same type as that for the Perisphere except that there was only one thickness of gypsum wallboard. The outer coverings of the Perisphere, Trylon, and Bridge were applied by using an exterior, steel-pipe, demountable scaffolding system, as shown in Fig. 17.

The interior shell of the Perisphere above the level of the light-projector platform was constructed with the use of a revolving scaffold made of tied-arch trusses having their curvature fitted to the inner Perisphere surface. The three fan-like trusses were interlocked with X-bracing. The scaffold was suspended from the zenith of the sphere by a spider-and-pivot hanging support. At the light gallery (100-ft level), the scaffold has a set of rollers, one under each truss, and these rollers run on a circular I-beam track. By means of horizontal platforms projecting out at various levels from the arched scaffold, workmen were able to apply a portion of a lune of inner surfacing and then move the scaffold around on its track into a new position, lock the rollers, and apply the surfacing to another lune. This procedure was repeated until the upper inner shell was completed.

The tied-arch scaffold framework, being bolted together, was easily dismantled upon completion of the surface, and, if necessary, can be put up again readily for maintenance purposes.

## Fabrication

Both the 1,000 tons of Trylon steel and the 2,000 tons of Perisphere steel were fabricated and erected by the American Bridge Company.

It was necessary to curve all members of the Perisphere, except the columns, webs of trusses, and interior framing, in order to fit the spherical surfaces. As a result of bending these members to arcs of great circles, as previously mentioned, the amount of work was held down to a minimum. More than one quarter of a million rivets, of which 100,000 were used during erection, were driven in order to fabricate and erect the 6.600 individual pieces of the Perisphere, varying in weight from a few pounds to 18 tons.

The curved members were bent cold by the use of jigs and bending machines. All curved trusses of the Perisphere were laid out and fabricated in specially designed jigs. The field connections in the meridian trusses were reamed assembled so as to minimize the inaccuracies and assure the best obtainable construction.

The bottom center drum and the ring girder, constituting the "nucleus" from which the Perisphere was constructed, are shown assembled at the fabricator's plant in Fig. 14. The ring girder was fabricated in eight sections and fitted to form a perfect circle 72 ft in diameter.

## Erection

*Trylon.*—The 615-ft Trylon was erected by means of a 70-ft basket boom. The column steel came in sections varying from 19 ft to 33 ft in length, and when the basket boom had surrounded itself with tower framework, it was lifted and the guys fastened in higher positions provided for in the column details. By this method the Trylon, including the top steel-plate section, was completely erected.

*Perisphere.*—A caterpillar-tractor crane having a 100-ft boom and a 12-ft jib was used to erect the Perisphere steel to within a short distance of the equator. Before reaching this stage of construction, however, the tractor crane set up an erection tower 30 ft square and 90 ft high on the inside of the Perisphere. The tower rested on, and was fastened to, the inner chords of the meridian trusses within the ring girder. Two 12-ton stiff-leg derricks set on diagonally opposite corners of the tower erected the remainder of the Perisphere with their 97-ft booms.

In general, the procedure of erecting the Perisphere was that of first placing one section of each of the meridians and then inserting the corresponding units of the girt trusses. Then the next sections of meridian and girt trusses were bolted up, and so the sequence was repeated until completion. The riveting followed the erection very closely so as to have all connections riveted before very much load came on the framework. No camber was provided in any part of the structure; consequently, the completed Perisphere has its natural dead-load deformation.

Due to the process of erection (placing a complete section of corresponding meridians and girts before proceeding to the next section) all members were erected in practically an unstressed condition. At some stages of the erection, the unsymmetrical temperature effects on the steelwork were enough to create gaps of $1\frac{1}{2}$ in. on closing units of girt trusses. This opening was taken care of by adjustment back into all of the thirty-two units of a girt truss so that in following such method no reaming of any connections was necessary in the entire erection of all the meridians and girts.

It is worthy of note that, although these structures were unusual in both type and details, no unforeseen difficulties arose, during either the fabrication or the erection, that involved delays or required revisions.

## Acknowledgments

The Theme Center was built by the New York World's Fair 1939, Inc., under the direction of the Department of Construction, with John P. Hogan, President, Am. Soc. C. E., chief engineer, and L. B. Roberts, M. Am. Soc. C. E., assistant chief engineer.

The design and plans of the steelwork for the Perisphere, Trylon, and Bridge were prepared by Waddell and Hardesty under the general direction of Mr. Hardesty, those for the Perisphere being supervised by Mr. Hedefine, and those for the Trylon and Bridge by Francis De Schauensee, M. Am. Soc. C. E. The foundations were designed by Moran, Proctor and Freeman, and the architects were Harrison and Fouilhoux. The original manuscript, complete with bibliography, has been placed on file for reference at The Engineering Societies Library, 29 West 39th Street, New York, N. Y.

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DISCUSSIONS

# THE RÔLE OF THE ENGINEER IN AIR SANITATION A SYMPOSIUM

## Discussion

By J. J. Bloomfield, Esq.

J. J. Bloomfield,[27] Esq. (by letter).[27a]—Messrs. Hatch, Griffin, Fair, Dyktor, Dunstan, and Pool have enhanced the original material in the papers presented by Professor Phelps and the writer by their excellent discussions and have brought out many important points bearing on the subject of the rôle of the engineer in air sanitation. The writer will not attempt to comment on their discussions but will confine his closing remarks to two points which, perhaps, need further emphasis. These points concern themselves with the preparation of an engineer for industrial hygiene work and the rôle of health department engineers in this field.

In the paper the writer discussed the type of training that an engineer should undergo in order to qualify himself for work in industrial hygiene. He also outlined those duties and qualifications of an industrial hygiene engineer which were developed by the Committee on Industrial Hygiene of the State and Provincial Health Authorities. This latter committee specified that the individual be a graduate in chemical engineering. However, in his version of the preparation of an engineer for industrial hygiene work, the writer stated that an individual with basic engineering training—in sanitation, mechanics, or chemistry—could become qualified (with special preparation) to conduct work in industrial sanitation. In view of the fact that industrial hygiene as practiced today includes all phases dealing with man's working and living environment, it would seem that an engineer with basic training in public health procedures would be best qualified to undertake this type of work, with additional training in those subjects outlined in the paper.

The other point which the writer desires to emphasize concerns itself with

Note.—This Symposium was published in November, 1930, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1940, by Messrs. Theodore Hatch, Donald Francis Griffin, Gordon M. Fair, and H. G. Dyktor; and June, 1940, by Messrs. Gilbert H. Dunstan, and Charles Lundy Pool.

[27] Sanitary Engr., U. S. Public Health Service, Washington, D. C.

[27a] Received by the Secretary June 12, 1940.

the necessity for engineers in public health work to recognize that industrial hygiene is as much a function of their activities as of the physician, nurse, or other public health worker. Industrial hygiene attempts to improve the hygiene of the individual and the environment in which he works and lives. The latter function belongs within the sphere of the engineer. Unfortunately, in the past, and to some extent even today, many engineers in public health work have confined their efforts to the control of the water supply, sewage disposal, foods, and milk, but have overlooked other problems becoming significantly important of late, such as air pollution and housing. Professor Phelps has defined public health engineering as the art of directing the forces and activities of Nature to the protection and improvement of the public health. This is a sound definition and engineers in public health work should seize the opportunities now before them and expand their activities to embrace all the problems in environmental hygiene, so as to include not only man's environment at home but also at his work place. Every engineering division in a health department—in a state, city, or county—should have personnel on its staff prepared to deal with the manifold problems of sanitation in industry. It is only by such an approach that engineers in public health may rightfully consider themselves as fulfilling their functions.

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DISCUSSIONS

# PROBLEMS AND TRENDS IN ACTIVATED SLUDGE PRACTICE

## Discussion

By Robert T. Regester, M. Am. Soc. C. E.

Robert T. Regester,[14] M. Am. Soc. C. E. (by letter).[14a]—The discussions of the writer's paper have contributed valuable supplementary information regarding activated sludge practice.

The comprehensive scope of the subject necessarily restricted its presentation by the writer to concise statements of fact with occasional interpretative comments. The effort was made to present an impartial summary of American practice since 1930. That period was chosen because it represented an outstanding recent era of advancement in design, expansion in plant construction, improvement of operation, and intensification of research activities. Both agreements and departures in design and operating practices, regardless of the respective merits, were noted in the paper. Advocacy of particular methods and innovations was avoided. Undue emphasis upon the developments in any particular locality was undesirable. The writer's aim was to stimulate the interest of sanitary engineers in the problems of activated sludge practice and to encourage their efforts toward its improvement and standardization.

It appears that Mr. Pearse may have viewed the paper primarily in the light of Chicago's practice and without regard to the writer's objectives. An appraisal was purposely omitted from the paper as the writer preferred a critical analysis and valuation of developments by the reader. Mr. Pearse elaborates upon certain aspects of activated sludge practice in the Sanitary District of Chicago and upon features of its plants. Also, he notes important pioneer contributions by the Sanitary District.

In referring to the North Side Works, he mentions that the average capacity of the plant was rated in 1928 at 180 mgd. For purposes of accurate record, it should be noted that this original rating (3)[14b] was 175 mgd (item 3, Table

Note.—This paper by Robert T. Regester, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Langdon Pearse, M. Am. Soc. C. E.; April, 1940, by Messrs. Frank C. Roe, and Gerard A. Rohlich and Clair N. Sawyer; and June, 1940, by E. Sherman Chase, M. Am. Soc. C. E.

[14] Cons. Engr., Baltimore, Md.

[14a] Received by the Secretary June 14, 1940.

[14b] Numerals in parentheses refer to corresponding references in the Appendix of the paper.

1). Parts of the works which could not be readily extended were designed for 1960 conditions.[15] However, it has been explained (3) that, because of the unusually rapid population growth of the North Side area during the 1920–1930 decade, the plant started with an average sewage flow of about 200 mgd, which was considerably in excess of its design capacity. Furthermore, the plant successfully carried the load. The original liberally designed aeration period and the foresighted hydraulic provisions made possible the newly adopted rating of 250 mgd with the addition of only final settling tanks. The twelve units added in 1937 (item 3, Table 4) represented an increase of 30% in the surface area of the plant's final settling tanks. On the other hand, the rated capacity of the plant was increased 43%. Therefore, on the basis of 20% return sludge, the mixed-liquor settling rate was increased from the original design basis of 1,180 gal per sq ft daily to 1,300 gal per sq ft daily for the rating of 250 mgd. Likewise, the mixed-liquor detention period in the aeration tanks was decreased from the original design basis of 6.3 hr to 4.4 hr. These supplementary data may give a clearer understanding of attending conditions under the present rating. Their inclusion in Table 1 would have unbalanced the comparative design data and possibly would have obscured the recent tendency toward shorter aeration periods as noted in the paper.

Mr. Pearse questions the correctness of the writer's description of the method of sludge handling adopted by the Sanitary District as a "use of waste heat." It is noted by Mr. Pearse that the heat recovered by burning the sludge (when burned) is used in drying the wet sludge cake and that the generation of steam depends primarily on the use of additional coal. This implies that at least some of the heat is also used for the generation of steam.

The following editorial comment[16] upon the new Southwest plant appears to share the writer's classification of the method:

> "By far the most significant feature, however, is the plan to use a sludge-incineration process for waste-heat utilization in conjunction with a steam-generating unit that will operate pumps, blowers and house generators. Disposal of sludge by dewatering and incineration as contemplated at Chicago seems at present to be the most promising solution of a troublesome problem, particularly when large volumes must be dealt with."

The following statement is also enlightening:[17]

> "The method of dewatering and incineration adopted by the Sanitary District has the advantage of year-round operation, requires less area and is lower in first cost than the more conventional method, thereby producing large savings in the total cost of the works. Lower operating costs are expected also. Further, by burning the sludge under the boilers which generate steam for plant power some of the heat value can be recovered from the sludge. Also dried activated sludge can be withdrawn for sale as fertilizer as desired."

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[15] "Engineering Works," *Booklet*, The Sanitary District of Chicago, August, 1928, p. 71.

[16] *Engineering News-Record*, Vol. 115, No. 6, August 8, 1935, p. 202.

[17] "Budget Report on Engineering Work—1938, The Sanitary District of Chicago," by William H. Trinkaus, M. Am. Soc. C. E., chief engineer, p. 11.

Further explanation is given as follows:[18]

> "Dried sludge averages 7,000 Btu per pound which is enough heat to evaporate the moisture in the sludge when the filters are operated to produce a filter cake with 80% moisture. At lower moisture content there will be some heat available for generating steam and at higher content some heat will have to be supplied by burning more coal.
>
> "Steam for plant operation will be generated by four water-tube boilers, each having maximum capacity to produce 110,000 lb of steam per hour at 425 lb gauge pressure and 735 degrees final temperature. Pulverized coal and dried sludge will be used for fuel."

The writer hopes that the foregoing quotations have answered in his favor the question which was raised. The real question seems to be how much power will be generated by the excess heat from the sludge when it is burned under the same boilers with additional pulverized coal. It is regretted that Mr. Pearse did not include the design heat-balance diagram for the plant in his discussion.

With reference to preliminary treatment, Mr. Pearse comments that the use of fine screens should have been noted in the paper. The existing installations of fine screens at Milwaukee, Indianapolis, Pasadena, and Charlotte (items 4, 7, 18, and 29) were noted in the footnotes of Table 1. Fine screens were not specifically discussed because to the writer's knowledge they have not been installed in any activated sludge plants, of large and medium capacities, constructed in the United States since 1930. However, in some instances consideration was given to the use of fine screens,[19] but they were rejected in favor of the more effective preliminary settling tanks.

Mr. Pearse confirms the writer's personal views with regard to the desirability of providing preliminary sedimentation. In addition, Mr. Pearse has offered an excellent reason for including preliminary settling in the Calumet and Southwest works which was not mentioned in the paper—that is, a mixture of fresh and activated sludge will dewater more readily on a vacuum filter than activated sludge alone, as shown by Chicago tests.

Both Mr. Pearse and Mr. Roe have noted the absence in the paper of a specific discussion relating to porous diffusers, except under "Needed Study and Research." The omission of that topic was intentional as diffusers had been recently discussed before the Society by S. W. Freese, M. Am. Soc. C. E. (12), who noted the related problems and trends.

The writer mentioned that blow-off valves, for either the air discharge manifold or the discharge pipe of each blower, were a new feature of centrifugal blower installations. Mr. Pearse stated that such use for blow-offs is almost unknown, and indicated that a blow-off for rotary positive-displacement blowers may be highly desirable. The writer was not referring to the pressure-relief valves as commonly used with rotary blowers. On the contrary, he mentioned that blow-off provisions are useful in minimizing the "pumping effect" when a

[18] "Southwest Treatment Works of the City of Chicago," by William H. Trinkaus, *Municipal Sanitation*, Vol. 10, No. 4, April, 1939, p. 231.

[19] See, also, report, "Proposed Sewage Treatment Plant at Ward's Island," by George W. Fuller, consulting engineer, Board of Estimate and Apportionment, City of New York, 1928, p. 88. See also reference (8) in the Appendix of the paper.

centrifugal blower is being added to those already in operation. The necessity for this provision is stressed by at least one of the four prominent manufacturers of centrifugal blowers. Another of these firms, which has furnished many of the blowers for the Sanitary District, having straight, radial-blade impellers, finds it unnecessary to have a blow-off valve. At Columbus, provision was made in the design of the blower piping for adding a blow-off valve on the discharge pipe of each blower to satisfy the manufacturer first mentioned, in the event that his units should be obtained as the result of competitive bidding. Actually, blowers of the straight, radial-blade type were installed. However, a blow-off from the discharge header will permit the system pressure to be lowered by exhausting to atmosphere, when desired. In the Cleveland (Easterly) and Baltimore (Back River) plants (items 6 and 9, Table 3), centrifugal blowers were furnished by the same manufacturer who desired the blow-off provision. In the latter case, the writer knows that a blow-off valve was installed for each unit at the insistence of the manufacturer. Apparently, the inherent pressure-capacity characteristics for a particular blower design dictate the necessity for blow-off valves so as to avoid the severe disturbances in the connected system when a blower is started and passes through its inherent unstable zone of operation (at about 50% of rated capacity) where "pumping" or "surging" occurs.

The writer was aware of some of the final sedimentation tests referred to by Mr. Pearse, and had studied the results. It was hoped that part of these data might appear as published contributions to the knowledge pertaining to activated sludge practice. The writer recommended further investigation of the efficiency of final settling tanks and factors affecting their design, and he continues to stress the need for such investigations.

Since the paper was presented, test data on concentrating activated sludge with a continuous feed centrifuge, of the type as installed at Columbus and Lansing, have been published.[20] These tests were conducted at Peoria, and showed that the centrifuge is capable of increasing the concentration of waste activated sludge from 1% to 5% solids. For that range of concentration, the total cost was estimated at $5.79 per ton of dry solids. In connection with the design of the Baltimore plant, the writer observed carefully conducted tests by personnel of the City of Baltimore (in cooperation with the manufacturer) of an experimental unit at Lima in 1937. Certain mechanical difficulties were troublesome at that time, but it was expected that these might be remedied later. However, as a result of these tests it was decided not to install centrifuges for activated sludge concentration at the Back River plant (item 9, Table 5).

Mr. Pearse also refers to the writer's statement regarding the probability that increased depth of sludge blanket produces a greater density of sludge. It is unfortunate that the 1931 test data of the Sanitary District, mentioned in the discussion, were not included. The writer had in mind a relatively small variation in the density of activated sludge with increased depth. He has viewed the results of certain tests which had shown some increase in the density

[20] "Concentrating Activated Sludge with a Continuous Feed Centrifuge," by L. S. Kraus and J. R. Longley, *Sewage Works Journal*, Vol. 11, No. 1, January, 1939, p. 9.

for sludge depths up to 10 ft. The value of such slight increases in sludge density can be appreciated by referring to Fig. 3. If the solid content of the sludge is increased from 1.0% to only 1.5%, it will be noted from Fig. 3 that the rate of sludge returned (to maintain 2,000 ppm of suspended solids in the mixed liquor) can be decreased from 25% to 15% of the corresponding sewage flow with consequent saving in return-sludge pumping costs. The statement did not refer to the concentration of activated sludge by either thickening tanks or centrifuges to perhaps 4 or 5% solids.

Besides discussing the interesting developments in the evolution of porous diffusers, Mr. Roe mentions the recent use of diffuser tubes in preference to plates for the activated sludge plant recently constructed at Gary, Ind.[21] For that 40-mgd plant, the designers selected stationary tubes, each 3 in. internal diameter by 24 in. long, which are set 24 in. above the floor of the aeration tanks. They are staggered along the submerged air header to give a spacing of 1 ft 4 in. A permeability rating of 40 was used. This development in diffuser practice for spiral-flow tanks in a large plant is significant in view of the fact that plates have given satisfactory service in numerous plants for relatively long periods of time.

Messrs. Rohlich and Sawyer discuss their findings from a praiseworthy investigation of the influence of temperature upon the rate of oxygen utilization by activated sludges. They also discuss the combined effect of suspended-solids content and temperature upon oxygen utilization as shown by their studies. It is hoped that the relationships which they express will be tried and observed in plant operation. Basic research of that character should aid materially in improving activated sludge practice, and should be encouraged.

Mr. Chase wisely refers to the caution that is necessary in the application of the results of laboratory research to the practical problems of design and operation. The conclusions from small-scale experiments, often conducted under artificial conditions, may be misleading unless proved under conditions of normal plant operation. In presenting the suggestions for needed study and research, the writer endeavored to interest plant operators, as well as research workers, in the possibilities for the further investigation of certain phases of activated sludge practice.

The basic design data for the 2.0-mgd plant at Leominster, as presented by Mr. Chase, is a valuable adjunct to the data given in the paper. The writer is pleased that a small activated sludge plant received deserving attention in the discussion.

Mr. Chase raises an important question when he suggests that some of the older tests may also be valuable indexes of the degree of purification obtained in activated sludge treatment. This matter should receive the attention particularly of plant operators and chemists.

In conclusion, the writer is gratified to learn from the discussions and from letters which he has received that the data presented in the paper may be found useful. He extends his sincere appreciation to all who have contributed to the discussions.

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[21] "Sewage Treatment at Gary," by L. R. Howson, M. Am. Soc. C. E., *Sewage Works Journal*, Vol. 11 No. 6, November, 1939, p. 994.

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DISCUSSIONS

# EFFECTS OF RIFLING ON FOUR-INCH PIPE TRANSPORTING SOLIDS

## Discussion

By G. W. Howard, Jun. Am. Soc. C. E.

G. W. Howard,[9] Jun. Am. Soc. C. E. (by letter).[9a]—The success or failure of any development in the laboratory is determined by trials in practice, and in discussing the field tests of rifled pipe it is desired to mention again the conclusions that were drawn from the laboratory studies. These are:

> "(1) Rifling will increase the efficiency of the line when the following materials are transported: (*a*) Coarse sand, and (*b*) gravel.
>
> "(2) Rifling will reduce the efficiency of the line when the following materials are transported: (*a*) Silt, and (*b*) clay."

As shown in the photographs presented by Mr. Brown (Fig. 12), the material in field tests was more evenly distributed in the rifled pipe than in the plain pipe. Results of laboratory tests, shown in Fig. 7, indicated that this condition would obtain. The relation of the "blocking" points between mixer types 8 and 12 also is of interest, but, as was pointed out, the mixing increase of type 8 over type 12 was not sufficient to justify a further reduction of the pipe diameter.

The field test described by Major Neuman was of particular interest because it was made with a dredge pumping the type of material with which the rifling would not increase the efficiency of the discharge line. It is of interest to note that the rifling allowed passage of large clay balls, but was plugged, at a later date, when the dredge was working in sawmill slabs and roots. No comparison of plain pipe was made in these sawmill slabs and roots, but it was pointed out by Major Neuman that, in the opinion of the dredge operator, there were no more plugs than would have occurred if a plain discharge line had been used.

The data presented by Mr. Newell include basic information on a complete field test. It should be noted, however, that the conditions under which the

Note.—This paper by G. W. Howard, Jun. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Fred R. Brown, Jun. Am. Soc. C. E.; April, 1940, by David L. Neuman, M. Am. Soc. C. E.; and May, 1940, by R. Y. Newell, Jr., Esq.

[9] Associate Engr., Office, Chief of Engrs., U. S. Army, Washington, D. C.

[9a] Received by the Secretary July 12, 1940.

pipe was tested were the most favorable of all that were encountered in the various field tests. Later tests by the Memphis Engineer District using the dredge *Burgess* did not show favorable results, although it was not shown that rifling reduced the efficiency of the line. These tests were made in a much finer material. Fig. 15 shows the mechanical analyses of material pumped in the tests on the dredges *Jadwin* (Mississippi River, Cedar Point Bar), *Burgess* (Mississippi River, Mile 230), and *Henry Bacon* (Savannah River). It can be seen that only the dredge *Jadwin* was working in material classified as coarse sand.

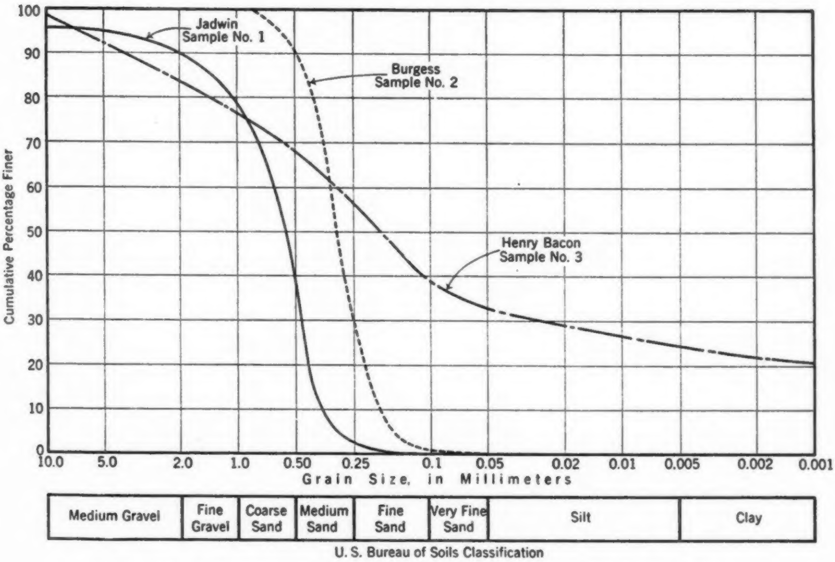


FIG. 15.—REPRESENTATIVE SAMPLES OBTAINED DURING PIPE LINE TESTS

Short tests, such as the three mentioned, each of which lasted about one week, cannot be considered as conclusive. They do indicate, however, that rifling is of value under certain field conditions.

A test which lasted for two years was completed by the Memphis Engineer District.[10] This test consisted in equipping one dredge with rifling and comparing its output record with that of another dredge of identical construction. Chief conclusions from this study of production data were:

(1) Increase of production (14.7%) for dredge with rifled line;
(2) Decrease of fuel costs (9.5%) for dredge with rifled line; and
(3) Decrease of total costs ($0.0038 per cu yd) for dredge with rifled line.

The elapsed time of this test is such that the data should be considered of more significance than for the other studies. The fact that the test covered all

[10] "Rifling in Dredge Pipe Lines," *The Experiment Station Bulletin*, Vol. 2, No. 1, February 1, 1939, pp. 1–5.

operating conditions encountered by the dredges during two years also is of significance.

In closing it is desired again to emphasize the fact that rifling the discharge line of hydraulic dredges is not a "cure-all" for output problems. Laboratory tests, confirmed qualitatively by results of field tests, however, have indicated the validity of the following criterion:

> Rifling in the discharge line of a dredge will increase the efficiency of the line in cases where the material being dredged through a plain pipe would settle along the bottom in appreciable quantities.

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# DISCUSSIONS

# WATER SUPPLY ON UPPER SALT RIVER, ARIZONA

## Discussion

BY JOHN GIRAND, ASSOC. M. AM. SOC. C. E.

JOHN GIRAND,[16] Assoc. M. Am. Soc. C. E. (by letter).[16a]—The need for knowledge concerning future runoff probabilities in power estimating is questioned by Mr. Wood, who states: "Provision should be made for the variations that surely will come by applying suitable contingency factors * * *." Just what constitutes a "suitable" factor, or how such a suitable factor is obtained, is not stated. Although a "suitable" factor might be obtained on the Tennessee River, with 70 years of gaging records, the use of such a factor on the Upper Salt River with only 15 years of record might result in wide differences of opinions among engineers.

Fig. 7, the contour map of runoff of the Tennessee River, undoubtedly is useful in its place, but cyclic analysis requires smoothing of the hydrograph over long periods of time, rather than breaking up the hydrograph into magnitude parts for short periods of time. Under such conditions a contour map of runoff could scarcely be expected to be anything except irregular.

Mr. Sherman "leavens the loaf" with the statement that "* * * the sunspot theory shows remarkable correlation with precipitation after both events have happened." Such criticism might have been devastating had it not been followed by a quotation from the paper[13] of Sydney M. Wood giving forecasts of the elevation of the surface of Lake Michigan, based on the sunspot theory, and Mr. Sherman's admission that "With a few examples, like the foregoing, engineers will begin to have some confidence in sunspot forecasting."

For comparison, it is interesting to note that the results obtained by the methods outlined in the paper were about 10% lower than results obtained by

NOTE.—This paper by John Girand, Assoc. M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Dana M. Wood, M. Am. Soc. C. E.; April, 1940, by LeRoy K. Sherman, M. Am. Soc. C. E.; and June, 1940, by George F. McEwen, Esq.

[16] Asst. Hydr. Engr., James B. Girand, Phoenix, Ariz.

[16a] Received by the Secretary June 19, 1940.

[13] *Bulletin*, Associated State Eng. Societies, Chicago, Ill., October, 1936, pp. 83–102; see Fig. 1, p. 87.

orthodox methods; this constitutes an additional safety factor, and is in line with operating experience in this area.

Professor McEwen has presented the crux of the matter in his statement, "* * * statisticians have long recognized that time series cannot be dealt with by the ordinary statistical methods because the observations ordered in time are in general not mutually independent or random in time." This fundamental fact has been neglected by many engineers who still consider day-to-day hydrograph data as independent figures, any group of which may be selected for analysis without regard to considerations of whether the group represents average conditions.

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DISCUSSIONS

# MEASURING THE POTENTIAL TRAFFIC OF A PROPOSED VEHICULAR CROSSING

## Discussion

BY MESSRS. CHARLES B. WINICK, AND JAMES H. S. MELVILLE

CHARLES B. WINICK,[6] ASSOC. M. AM. SOC. C. E. (by letter).[6a]—An able and logical exposition of the elements entering into the preparation of a survey which deals with the prediction of traffic for a newly proposed vehicular crossing is presented in this paper. It is a source of gratification to know that even so complicated a subject as traffic analysis, with all its accompanying ramifications, can be brought to some semblance of logical conclusions in the hands of a competent observer. The numerous factors, technical as well as otherwise, are so complex in a study of such character that one must be constantly on the alert in order to avoid the pitfalls of unsound and illogical conclusions which are likely to be drawn unless the accumulated data are thoroughly scrutinized and examined minutely, all their elements segregated, and their relations to the entire subject carefully studied.

A little reflection will reveal at once the importance of traffic analysis to the projection of a new vehicular crossing. Proper traffic analysis is the most important tool in the hands of the engineer, banker, industrialist, and politician before any proposed vehicular crossing may be either recommended or entirely rejected. Even after the project is recommended its exact location, kinds of approaches, connecting highways, future provisions for expansion, etc., must all be studied in the light of traffic analysis. Even the alinement and grade of the proposed crossing must be determined for the convenience of the motorist who is to use the facility and who in the last analysis is the initial unit of any traffic study.

It is incumbent upon the engineer charged with the responsibility for the construction of a large project to prepare all the necessary data in order to arrive at a proper recommendation as to whether such project is physically and economically sound, and what advantages, if any, it may have compared to

NOTE.—This paper by N. Cherniack, Assoc. M. Am. Soc. C. E., was published in February, 1940, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1940, by Murray D. Van Wagoner, M. Am. Soc. C. E.

[6] Engr., Queens Midtown Tunnel, PWA, New York, N. Y.

[6a] Received by the Secretary June 3, 1940.

another type of facility—say a bridge versus an underground tunnel. Will it be self-sustaining? Will it produce an unnecessary economic burden on the communities that are to enjoy the benefits of such facility, or will it enhance economic life? Will the architectural treatment of the facility become a ban or a boost? Will esthetic values be modified? Many other tangible and intangible factors must be known prior to the time the engineer is ready to argue his case meritoriously before any public body or financial institution which has an interest in the creation of such facility.

Once the type of crossing and its location are determined it becomes a relatively easy task to estimate within reasonable limits the cost of construction, cost of real-estate acquisition, administrative and engineering costs, etc., which enter into the total estimate of the project cost. After computing the operating and maintenance charges, the cost of amortization, interest, taxes, etc., the engineer is generally in an advantageous position to know, more or less with exactness, the amount of annual revenue required in order to meet all charges during the useful life of such project. The difficult task of determining the toll charge becomes the most important factor that will determine the feasibility of the project.

If the toll is predetermined (as is most often the case), the facility must be so designed and planned as to attract the required amount of traffic corresponding to the total required revenue. In cases where the toll charge is determined because of other free or competing facilities, or any other considerations, the most accurate traffic analysis will of necessity become prejudicial, since all interests connected with the promotion of the project will tend to develop some one or another thesis that the required amount of traffic will be met or surpassed. Since most traffic consists of diverted and generated components, both of an uncertain nature, particularly the "generated" portion, it can be seen readily that it is difficult for a conscientious and honest analyst to avoid the danger of being somewhat too optimistic; and for that reason an unbiased analysis by a competent observer disinterested in the project is always desirable.

*Use of Empirical Formulas.*—As is the case with any empirical formulas, the expressions of traffic growth are subject to criticism due to the variation of the several components that enter into making up such a formula. Whether the straight line, compound interest, or Pearl Reed curve is applied to estimate the annual traffic growth, accuracy is sacrificed at the expense of the several assumptions that are physical in nature and subject to wide variations. Thus, in the straight-line formula the assumption that the annual increment of growth is constant is obviously erroneous. It takes but little observation to conclude that the annual increment of growth is dependent on too many physical changes to remain constant for any considerable period of years. Similar objections exist in any of the other empirical formulas expressing traffic growth.

*Rating of a Proposed Crossing Based on Cost Differences.*—Again, in endeavoring to obtain the merit rating of any proposed crossing relative to existing crossings in the same "line of travel," the basic assumption of cost differences, monetarily evaluated, is subject to numerous variations. Taking the case of $\Delta T_w \Delta C_w$, which represents the product of the waiting-time difference via any

crossing compared to that via the "standard" crossing by the monetary evaluation of the waiting-time difference, in cents per minute, note the following:

The monetary cost of waiting time of a highly paid executive who is en route to negotiate an important business contract cannot be compared to that of the "young lover" who is leisurely taking out his girl friend for a joy ride. Time is of no essence to the latter and hangs heavily on the former. Although it is true that the average commuter's time, to and from work may be evaluated with some degree of accuracy, no such appraisal of the cost of time can be made for a patient who is hospital bound by ambulance for an emergency surgical operation which is a matter of life itself. The same considerations would also hold true of $\Delta T_r \Delta C_r$ representing the product of the running-time difference via any crossing compared to that via the "standard" by the monetary evaluation of the running-time difference in cents per minute. Many more examples can be cited, but the foregoing are sufficient to illustrate the differences arising in waiting-time or running-time costs between extreme cases; and therefore the application of a constant rate of cost for such factors to all types of motorists would obviously lead to some erroneous conclusion. It must always be remembered that a variation of any basic assumption in the rating formula affects the merit rating of the proposed facility, and students of traffic analysis should pause for a moment before accepting the author's recommendation of merit rating as a basis for predicting future traffic on a proposed crossing.

*Preference and Prejudice Factors.*—It is easy to draw a smooth curve between any two points and conclude that such curve is a function of the preference or prejudice factors of a given route along any given "line of travel." The contention that the "scatter" of the data points revealed by a plot of relative patronage against relative cost has any significance is questionable. Even if each motorist were stopped and questioned as to his reason for using a given crossing in preference to any other competitive crossing along the same line of travel, such data could not be applied to a forecast of probable traffic on a proposed new facility, for it does not follow that the same reasons under similar circumstances would motivate another motorist or even the same motorist to use the new facility.

*Relative Patronage Factors.*—Closer analysis of relative patronage factors would indicate that they are not only psychological and economic in nature but geographical and social as well. For instance, a facility built for the use of car owners representative of an agricultural or farming community would be affected quite differently from that of an urban community of the same population, largely because of the different habits and customs of the two types of communities. Again the social characteristics of a typical New England Yankee community, being entirely different from those of a community peopled by foreign-born elements, would result in marked variations of "relative patronage" for a given or newly proposed facility. Thus, it can be seen that it becomes extremely difficult to arrive at any rational figures which represent all the factors that contribute to relative patronage and relative cost data.

*Trip Cost.*—Were the average motorist to reflect a moment on the actual cost per trip that he ultimately pays for crossing a given facility, the engineer

would be in an eminently qualified position to actually calculate the volume of traffic with reasonable precision. It would take much effort, however, to educate the average motorist to the actual cost per trip since he is not readily receptive to any assimilation of statistical data which directly affect him. Even if an educational campaign were successful, the motorist would soon become cost-conscious, and this would result in putting most of the automobile finance companies out of business, causing a material reduction in traffic growth.

The author assumes a cost of waiting-time difference at 2 cents per min and running-time difference at 3 cents per min. Such assumptions are more or less arbitrary since the patronage of any facility is entirely dependent on geographic, psychological, social, and economic factors as stated previously. A variation, for example, of the waiting-time difference to, say, 3 cents per min instead of 2 cents would dislodge all of the figures as shown in Table 6 and would produce different relative patronage distribution of each facility on a given "line of travel." Therefore, basing the forecasting of traffic for any new facility on such arbitrary assumptions may lead to erroneous conclusions. Thus, it can be seen that even a neutral observer, representing the motorist and taxpayer and not the banker, builder, or financier, is confronted with the most difficult task when endeavoring to evaluate all of the elements entering into any equation contributing to the patronage factors of any proposed facility.

*Projection of Traffic Into the Future.*—It cannot be denied that the number and value of the merit rating of competitive crossings act as a barometer for the traffic distribution of all components along a given "line of travel." Were the merit ratings of such nature that their determination depended on fixed and tangible elements, such a method would offer a ready means of projecting traffic into the future. Unfortunately such method is open to the numerous objections previously mentioned, and, even in the hands of an expert analyst who is quite familiar and able to modify such intangible concepts, a quantitative forecast may fall short of actual performance and often does, as was the case with the Lincoln Tunnel.

Nevertheless such analytical data are of considerable importance qualitatively, since thay enable one to predict trends and traffic patterns. Furthermore, when upon opening the new facility, comparisons of actual with predicted traffic performance are made, noted discrepancies can be studied and accounted for in the light of changes that occurred between the time that the traffic studies were made and the time of the actual opening of the facility. Suffice it to say that the improvement of any competing facility prior to opening the new facility would upset any prediction favoring the latter. A case in point, as the author states, is the completion of the West Side express highway which shortened the time between the Holland Tunnel and the George Washington Bridge and affected the predicted traffic for the Lincoln Tunnel adversely. It also can be seen at a glance that an improvement at the Manhattan approach to the Queensboro Bridge would affect the traffic materially on the Queens Midtown Tunnel. Although the Queensboro Bridge is heavily traveled, no appreciable delays are encountered on the span itself. The Manhattan approach could be modified and modernized by: (1) Removing the Second Avenue Elevated Line; (2) demolishing a substantial number of buildings; (3) creating

a large plaza; (4) rearranging traffic signaling so as to permit traffic to proceed northward on Second Avenue; (5) building private streets where necessary; (6) eliminating or at least modifying the traffic light, or possibly reconstructing that part of the bridge which provides access to Welfare Island; etc. Under conditions of maximum improvements at both ends of the bridge the nearly "ideal" state of travel on such facility would be reached, and it could then be determined whether the bridge is overcapacitated or capable of absorbing additional traffic.

That a crossing is as good as ability of traffic to enter or leave it may be seen further from a survey of conditions of the East River bridges at the lower end of Manhattan. Those approaches were designed for "horse-and-buggy" days and are now entirely obsolete. Any improvement, therefore, on such approaches would soon be reflected in a traffic redistribution of all other crossings in the same "line of travel" relative to the ease and comfort, time saving, etc., that such improvement would afford. Thus the widening of Chrystie Street at the Manhattan end of the Manhattan Bridge has resulted in a marked increase of traffic on that bridge. It may be concluded that unless all competing facilities along a given "line of travel" are so constructed or altered as to receive traffic under ideal or nearly "ideal conditions any attempt to project traffic into the future" is vulnerable to a redistribution, commensurate with improvements of other competing crossings. Witness the hopeless congestion at Canal Street, in Manhattan, upon leaving the Manhattan Bridge. Surely a modification of that approach would have a marked effect on relative traffic patronage on any of the other bridges in that area. Again, the much talked of necessary improvement of the Brooklyn Bridge would affect a traffic redistribution on the lower bridges and any forecasting of traffic for the newly proposed Battery-Hamilton Avenue Tunnel (Manhattan to Brooklyn, N. Y.) would thereby become subject to considerable modifications and revisions.

*Relative Toll Charges.*—Prior to the fixation of a permanent toll charge for the purpose of raising sufficient revenue to insure economic practicability for the promotion of a contemplated crossing, several factors must be studied. After all, when the engineer compiles his estimate of the cost of construction, operation, maintenance charges, etc., he is reasonably certain as to how much gross income will be required in order to meet the necessary amortization, interest charges, taxes, cost of financing during construction, operating costs, maintenance cost, etc. Assuming an arbitrary toll charge, a certain minimum amount of traffic would be required to insure economic success of the project. Such amount of traffic is either in the proximity of the forecasted traffic as prepared from other characteristics or, if deficient, the toll must be increased to meet all the required charges. Once the arbitrarily assumed toll charge is increased an unbalancing of the forecasted traffic results. Such would be the case particularly when competing with free facilities. It thus may become a vicious cycle of increasing the toll and correspondingly decreasing the traffic that produces revenue. Thus, taking the case of either the Queens Midtown Tunnel or the proposed Battery-Hamilton Avenue Tunnel, it is safe to conclude that a toll charge of 10 cents per car would attract considerably more traffic than if the toll were, say, 25 cents or 50 cents per car. There is un-

doubtedly some point of balance at which the economic law of diminishing returns becomes operative. This point must be determined carefully in order to arrive at a maximum economic yield before any intelligent forecast of future traffic is finally determined. Such intangible assets as increased or decreased land valuation in the approach vicinity, industrial and commercial benefits, public convenience, etc., although difficult of economic evaluation, nevertheless affect markedly the determination of toll charges. The writer is of the opinion that a variable toll charge for different periods during the life of the facility is likely to result in a more uniform distribution of traffic than a charge fixed at the opening of the facility and remaining constant during its entire life.

*Generated Traffic.*—The writer is keenly interested in ascertaining whether there is any reliable, scientific method of forecasting the "generated traffic." Reliance on past experience and former observation is too uncertain, especially when it is realized how important a factor such an element may become. Frequently the introduction of a new river crossing at a certain location may result in a shift of the entire residential or industrial community at either one or both ends of the crossing, causing a corresponding change in traffic distribution generally and in the "generated traffic" particularly. Cases in point are the Holland Tunnel and, say, the Brooklyn Bridge. It was not so long ago that the area in the vicinity of the Manhattan approach to the Holland Tunnel was occupied by old, dilapidated, and unsightly buildings. No sooner had the tunnel been placed in operation than active construction of commercial, industrial, and even residential buildings not only transformed the slum and blighted areas in the approach vicinity into modern developments, but such activity necessarily resulted in a spurt of generated traffic. In contrast, the real estate in the "shadows" of the Brooklyn or Manhattan bridge approaches has depreciated constantly and property owners are forced to neglect maintenance and repairs. This causes obsolescence and unsightly ugliness, and is creating one of the worst slum areas in the city. In such cases the "generated" traffic is continuously declining. Under such extreme circumstances, how is any one to arrive at a rational method of predicting the annual increment of generated traffic?

It may very well be that more careful attention to esthetic values will result in better architectural treatment of future crossings, whether it be a bridge or a tunnel, and the resultant upheaval of real estate will be minimized. It is hoped that future approaches will be so treated as to enhance the property values of real estate, enabling the property owners partly to finance the cost of such improvement.

*Traffic Determinants.*—The determination of future traffic by such determinants as gasoline consumption, department store sales, payrolls, purchasing power, or what have you, is as good a method as is a forecast based on past performance; but even such determinants are subject to wide and unforeseen variation. It may very well occur that the new crossing as projected may serve an industry which may thrive, despite economic depressions or recessions. Department store sales, the total consumption of gas, and all such other determinants may take a decided drop when computed for the entire traffic

reservoir. Nevertheless the traffic for the newly projected facility may still show an upward tendency; or the reverse may be true—that is, while all traffic determinants may increase, traffic for a single given facility may show a decrease. In short, it would seem unsafe to forecast traffic for a newly proposed facility on such determinants.

*Analysis of Traffic on Streets in the Proximity of the Plazas.*—The assertion that the increase of traffic on the city streets due to a newly constructed adjacent plaza from a crossing may be forecast with a reasonable degree of accuracy is open to the following objection. Experience will show that motorists accustomed to use a certain route when traversing the city, if unduly held up because of additional traffic from an adjacent plaza or if delayed on account of increased traffic from whatever cause, will soon abandon such route and take the adjacent streets paralleling the customarily traveled route. A new redistribution of traffic in the vicinity of the approach streets would soon occur, and the forecasted results for increased traffic for such streets due to the addition of the new facility would become entirely unbalanced. If the entire area in the vicinity of the new facility is already suffering from excessive traffic, it is only reasonable to conclude that provisions to absorb the additional traffic from the new facility must be made either in the streets immediately contiguous to the plazas or to adjacent parallel streets. Of course, a solution may also be found by improving traffic conditions on the existing streets by elimination of parking, alteration of signaling, or numerous other devices which will facilitate the movement of existing traffic.

In view of the nature and complexity of the subject the author's attempt to standardize and segregate all the elements affecting traffic is a worthy contribution to such a highly controversial subject. The mere recognition of all elements affecting traffic is a difficult task in itself. The author's logical treatment, simplicity and brevity of the theoretical analysis, and the various empirical formulas offered for use, as well as the ready applicability of the several methods suggested to the solution of practical problems, are worthy of high commendation. Mr. Cherniack is indeed to be congratulated for his effort to lighten the burden of engineers who are always in a dilemma when a new crossing is projected, lest traffic predictions may be insufficient to proceed with the planning of the new facility. The importance of such traffic data, even if not entirely quantitative, cannot be overemphasized. Undoubtedly other means will be found ultimately to further the accuracy of the author's proposed method for estimating traffic. Even new concepts and ideas may be developed, taking into account the possibility of changes in modes of traffic, modification of existing vehicles, road changes, urban and suburban population, traffic redistributions, effect of aerial travel development, etc. The author's development of the subject with the present meager data at his disposal should act as a stimulant to other engineers engaged in such fields, for it is only through the constant accumulation of more and more reliable data that traffic engineers may ultimately avert the many "traps" open to a study of such type. It is indeed a source of gratification to feel that the "right start" in the "right direction" has been made.

JAMES H. S. MELVILLE,[7] M. AM. SOC. C. E. (by letter).[7a]—Mr. Cherniack is to be congratulated on having written what is probably the first comprehensive paper on the nebulous art of estimating the probable traffic and revenue for a projected toll facility. His exhaustive review of the steps necessary for the formulation of a proper judgment on the part of the engineer in charge of such work has been ably presented and leaves little room for comment or criticism. Traffic engineers will welcome the fact that he has made generally available some of the valuable data on traffic and traffic habits collected over many years in the laboratory of The Port of New York Authority.

Although the majority of his data and the analyses drawn therefrom are directly applicable to the metropolitan area of New York City, and to traffic in that and other cities of a comparable nature, the principles enunciated and the conclusions drawn are applicable to any situation, provided the necessary data as to traffic volumes, origins and destinations, rates of growth, and relative costs of travel can be determined adequately for the location in question. The formulas developed by Mr. Cherniack in the Appendix of his paper are interesting and their reliability will doubtless be tested by application to specific instances in the future. Since one of the major problems in making estimates of prospective toll traffic is the correct interpretation of the available data pertaining to existing traffic and its habits, any mathematical formula which will give even a "first approximation" will be welcomed by traffic engineers as a concrete factor by which their own judgment can be checked and possibly corrected—especially when the formula is based on long and intelligent study of traffic habits and preferences in such a diversified area as the metropolitan district of New York City.

Engineers engaged to report on the traffic and revenue possibilities of toll facilities are usually handicapped by the inadequacy of the time available for the making of proper traffic studies. As a general rule, three to four months is the maximum time in which field work, tabulations, analyses, and conclusions have to be completed. For an important project only one set of field observations can be made, tabulated, and analyzed in that period of time. Unless there are existing toll facilities in the locality from which seasonal traffic patterns can be obtained, this is too short a time in which to obtain them independently, and the lack of local data similar to that contained in Mr. Cherniack's paper makes it difficult to supplement field observations and to formulate reliable premises for the foundation of estimates. It is hoped that the publication of this paper will give an impetus to the publication of further studies in different localities by representatives of public agencies.

[7] Amberley, Newcastle, Jamaica, B. W. I.; formerly with Coverdale & Colpitts, Cons. Engrs. New York, N. Y.

[7a] Received by the Secretary June 6, 1940.

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DISCUSSIONS

# TRANSIENT FLOOD PEAKS

## Discussion

By FRANKLIN THOMAS, M. AM. SOC. C. E.

FRANKLIN THOMAS,[52] M. AM. SOC. C. E. (by letter).[52a]—The combination of circumstances which produced the "New Year's Day Flood" of 1934 is of such rare occurrence that a repetition in the same general region is almost outside the range of reasonable probability. However, in presenting for reference through this paper his description of the phenomena which developed, and his analysis of them, Mr. Lynch has expanded the horizon within which engineers should direct their studies when considering the nature and scope of protective works to care for runoff from watersheds of steep gradient.

In a great majority of the years, the initial rains of the season on the mountain watershed concerned in this paper occur in October or November in sufficient amount to dispel the summer-long fire hazard for the brush covering of the mountain sides. During the autumn of 1933, the rainfall as recorded by the Pasadena (Calif.) station, which is representative for the watershed, was 0.48 in. in October, 0.20 in. in November, and 10.34 in. in December. The devastating fire which was a major contributing cause of the flood denuded the mountain sides over a 5-mile front on November 21, 1933, an exceptionally late date for such an extensive fire.

It can be stated as almost axiomatic that no floods will develop from the mountain watersheds of Southern California until a rainfall of heavy intensity, extending over several hours, occurs subsequent to an aggregate rainfall exceeding 10 in. within the previous two weeks which has saturated the watershed. Such conditions seem to occur about once in a decade in the region under discussion, but only twice did they occur during the 60-yr period of record earlier in the season than the latter part of January. Those instances were in December, 1889, and December, 1921. Although the December storm which caused the local flood described by the author did provide the basic precipita-

NOTE.—This paper by Henry B. Lynch, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Ivan E. Houk, M. Am. Soc. C. E.; March, 1940, by Messrs. Gordon R. Williams, and Donald M. Baker; April, 1940, by Messrs. James M. Fox, F. C. Finkle, A. L. Sonderegger, and Harold C. Troxell and R. Stanley Lord; May, 1940, by Messrs. Karl J. Bermel, and R. W. Davenport; and June, 1940, by Walter J. Wood and Maxwell F. Burke, Assoc. Members, Am. Soc. C. E.

[52] Prof., Civ. Eng., California Inst. of Technology, Pasadena, Calif.

[52a] Received by the Secretary July 15, 1940.

tion of 10 in., there was lacking the supplementary rainfall to produce a flood of serious magnitude from unburned areas.

The freshly burned area, lacking the protective cover, yielded a much earlier, as well as much greater, runoff than a corresponding surface in natural condition produced. The storm was one of sustained precipitation for at least fifteen hours before the burned watershed surfaces melted away into the canyons and succeeding washes. During the afternoon of December 31, recording gages in Pasadena showed a uniform unvarying rainfall of 1 in. per hr for three successive hours, with no cessation of earlier and later intensities which were almost as heavy.

The conditions of this storm were markedly different from those which caused the Utah floods of 1930 following abrupt precipitation of high intensities. The warning to engineers which this New Year's storm presents with its huge mass of water and debris is significant for its quantitative excesses. It compels a new approach to consideration of drainage openings, channel capacities, and probable life of reservoir space.

It is unfortunate that so much determination of what actually took place must be dependent upon hypothesis and conjecture. The author's support for his explanation of the surge effects in excess of the sustained runoff is well presented.

Other aspects of the situation, either involving debris in rapid movements along the channel, or debris which is being gradually shifted about in the channel, offer alternative bases for interpreting the phenomena described and point out new directions for observation and study.

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DISCUSSIONS

# AXIOMS IN ROADWAY SOIL MECHANICS

## Discussion

By Victor J. Brown, Esq.

Victor J. Brown,[6] Esq. (by letter).[6a]—What Mr. Porter has to say about one's first impression when beginning a study of soil mechanics (see heading "Definitions") is undoubtedly true. Practical usage has done much already to simplify the understanding of basic principles. Continued application of standardized test procedures to practical construction, followed by investigation and report, is also necessary if soil analysts are to reduce the subject to simple terms for more general understanding.

Mr. Porter has proposed a nomenclature which, the writer believes, should be adopted as rapidly as possible. Engineers speak loosely of a "fill," an "embankment," or a "foundation." The writer has many times heard the substructure referred to as the "subgrade." As this term indicates, a subgrade is a grade (a line or a plane) beneath something—a pavement, for example. Depending upon the type of construction, the wearing surface may have a different subgrade from the base, or the subbase.

After reading the thirteen axioms listed by Mr. Porter, the writer is left with one predominant thought; that is, that uniformity is a basic principle in dealing with soils—uniformity in moisture content, or uniformity in soil masses, or uniformity of various types of soils in separate layers. To the extent that uniformity is a desirable condition, investigation, testing, and design are important. In construction, one way to obtain uniformity of soil in the substructure is to mix all soil during excavation in the cut. In a deep cut, this uniformity is easily obtained by shooting the full face of the cut and then loading the loose, mixed material for haul to the embankment substructure.

When the type of construction equipment available can more economically excavate and dump in layers, construction procedure should be such as to place similar soil types in uniform layers, all uniformly compacted. Uniform compaction is as important as placing excavated material in uniform layers, whether compacted with Proctor optimum water content or something less.

Note.—This paper by Henry C. Porter, M. Am. Soc. C. E., was published in February, 1940, *Proceedings*.

[6] Publishing Director, *Roads and Streets*, Gillette Publishing Co., Chicago, Ill.

[6a] Received by the Secretary June 13, 1940.

Desirable density may be obtained with less than Proctor optimum water content provided the proper energy is expended on compaction either with heavier compacting rollers or more passes of the average compacting rollers. In compacting the soil substructure, the writer feels that it can be compacted too densely unless provision is made to positively prevent either increase or decrease of the contained water content—that is, prevent any change in water content after final compaction.

Several contractors have obtained uniform optimum water content by jetting the deposit from which excavated substructure soil is taken. This works particularly well when excavation is taken from borrow pits.

Mr. Porter stated one point, in Axiom 8, which deserves consideration to a much greater extent than merely a mention. The writer was pleased to note that he expressed the desired goal in terms of a desired uniform density rather than expressing compaction in terms of a certain amount of work to be done by a certain type of equipment. Too often specifications stipulate what volume of work is to be done by a particular type of equipment rather than leaving the selection of procedure for obtaining uniform density to the contractor, and specifying a required density. By stipulating what equipment shall be used and how, engineers are throwing obstacles in the path of progress for soil mechanics. Rather, they should specify a required density or state of compaction and allow construction men to devise their own methods for obtaining the required results.

To Axiom 13 the writer would add (*c*) to the effect that uniform compaction to a certain density or to a particular compaction condition must be specified.

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DISCUSSIONS

# FOUNDATION EXPERIENCES, TENNESSEE VALLEY AUTHORITY

## A SYMPOSIUM

### Discussion

By Messrs. V. L. Minear, and C. E. Blee

V. L. Minear,[28] M. Am. Soc. C. E. (by letter).[28a]—The Symposium on Foundation Experiences of the Tennessee Valley Authority is a noteworthy contribution to the literature on foundation treatment. The different papers not only describe in detail the successful methods used in treating the difficult dam foundations built upon, but, what is of equal importance, they open up for discussion various bothersome questions on foundation treatment in general, which it is believed could be considered with profit.

Among the debatable questions brought up, particularly in Mr. Lewis' excellent paper, are the following:

(*a*) What pressures shall be used in introducing the grout?
(*b*) What sizes and spacings of grout holes shall be used?
(*c*) How much and what kind of grout shall be injected?
(*d*) What methods shall be used?

The writer submits herewith his views on the foregoing questions in so far as they apply to Mr. Lewis' paper. These views, gained from experience in treating the foundations of twelve different dams, are offered with no thought of establishing authoritative rules; rather it is hoped to learn from the experience of others by stimulating discussion.

*Pressures.*—What pressures shall be used? This is a matter of prime importance as it influences the optimum spacing of holes and the water-cement ratio of the grout to be injected. The highest pressure that can be used safely is desirable, but, unfortunately, there is no safe rule for its determination. For lack of a better criterion some have used an arbitrary value of

Note.—This Symposium was published in March, 1940, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: May, 1940, by Messrs. George K. Leonard, and F. B. Marsh; and June, 1940, by Messrs. Berlen C. Moneymaker, R. F. Walter, William F. Prouty, Jacob Feld, and A. Warren Simonds.

[28] Drill and Grout Foreman, The Panama Canal, Diablo Heights, Canal Zone.

[28a] Received by the Secretary June 12, 1940.

one or more pounds of pressure per foot depth of hole. This rule is of doubtful value except in special cases. A more rational rule would be to require the application of a specified minimum pressure to all holes. This should be based upon the hydrostatic pressure to which the finished structure will be subjected. To this irreducible minimum, as much additional pressure should be applied as can be used safely. Not only does safe pressure vary with different formations, but it varies greatly from hole to hole in the same formation.

It has been observed that rock displacement does not ordinarily occur on those holes which take initial grout slowly and at high pressure. It is more often the one which consumes it freely for a considerable time that terminates in a bad grout leak indicating rock displacement. This occurs in many cases before the specified maximum pressure has been applied. Consequently, if damage is to be avoided and at the same time effective grouting done, some method other than the application of a uniform maximum pressure to all holes must be used. The use of "upheaval gages" described by Mr. Lewis is a step in the right direction. Some such instrument, more sensitive than the ordinary engineer's level, is fully as essential in foundation grouting as are the dial gages or extensometers used almost universally across the contraction joints of masonry dams when they are grouted.

A valuable adjunct to rock grouting, intended to serve the same purpose as upheaval gages, is the portable tiltmeter devised by R. S. Lieurance. This instrument consists of an optical lever containing a pool of mercury which is capable of indicating extremely small movements due to tilting. When such movements occur, the pressure of injection can be reduced accordingly before damage is done. The instrument is light, sturdy, readily transported, and requires no special knowledge for its operation. Its principal advantage lies in the fact that it obviates the necessity for drilling holes and grouting in rods as is required when upheaval gages are used.

There are two criteria in common use for the final application of pressure to foundation holes. The first requires that pumping continue until the hole refuses to take 1 cu ft of grout in some specified time (such as 10 min) at the specified pressure. The second criterion is that injection shall continue to refusal at a pressure which is a fractional part (such as two-thirds) of the allowable pumping pressure. The writer prefers the latter method. He and other engineers experienced in this line of work believe that requiring the contractor to pump on each and every hole until that hole consumes less than 1 cu ft of grout in, say, 10 min of pumping, works a hardship on the contractor with but little benefit to the owner. They argue that it is as necessary to keep grout agitated in the pump and delivery line as it is in the sump if cement is to be prevented from settling out of the grout and that when grout is being introduced at so low a rate, the velocity is so low that most of the cement settles out of the grout stream and finds lodgment in the pump and delivery line while the injectamenta is largely water. It is thought that, so far as the quality of the work is concerned, the foregoing are matters of little moment; that the hole has been grouted, successfully or otherwise, long before the final pressure is applied; and that the selection of a proper water-cement ratio, at

the proper time, is of vastly more importance than the injection of the ultimate cubic foot of liquid into what is probably a plugged hole.

*Hole Layout.*—What size and with what spacing shall grout holes be drilled? This is a question of fundamental importance since the drilling of holes constitutes a major item in any rock grouting program. The cost of such drilling is ordinarily influenced by the diameter and depth of the holes. This is particularly true when the condition is such that diamond-set bits must be used. The unit prices in Table 4, which were bid on a large western dam, are typical of this tendency.

TABLE 4.—UNIT BID PRICES, IN DOLLARS, FOR GROUT AND DRAINAGE HOLE DRILLING

| Diameter of holes (inches) | DEPTHS IN FEET | | | | | |
|---|---|---|---|---|---|---|
| | 0 to 30 | 30 to 50 | 50 to 75 | 75 to 100 | 100 to 150 | 150 to 200 |
| $1\frac{3}{8}$ | 1.40 | 1.70 | 1.85 | 1.90 | 2.00 | 2.10 |
| 3 | 2.50 | .3.00 | 3.00 | 3.25 | 3.50 | 3.50 |
| $5\frac{1}{2}$ | 5.75 | 5.75 | .... | .... | .... | .... |

Under similar conditions, for any given sum of money that can be expended in drilling, more holes and consequently a closer spacing can be used when small-diameter holes are drilled. Since the goal of rock grouting is to fill the rock fissures, and since the pattern which these fissures follow is unknown, it would seem that the closer the spacing of holes, the greater the probability that all seams will be filled and hence that that diameter of hole is most desirable which can be drilled most cheaply, provided that the cheap hole allows grout to be introduced as effectively as does the more costly hole.

The quantity of grout that must pass each unit area of the hole's cylindrical surface is greater in small holes than in large holes, and therefore it might be expected that small holes would tend to become obstructed. This is true. The writer has noted results similar to those mentioned by Mr. Lewis under "Curtain Grouting: Action of Grout." However, this characteristic does not justify the drilling of oversize holes, especially in the light of experience. Data on the record hole on each of three large dams constructed during the decade 1930–1940 are as follows:

| Dam | Hole diameter (inches) | Solids injected (cubic feet) |
|---|---|---|
| *A* | 1 | 5,000 |
| *B* | $1\frac{7}{16}$ | 28,800 |
| *C* | $1\frac{1}{2}$ | 38,000 |

If 28,800 sacks of cement can be introduced through a single $1\frac{7}{16}$-in. hole, the necessity for drilling larger holes is not apparent. It would seem that the quantity of grout that can be injected depends more upon the structure penetrated than it does upon the diameter of the hole drilled. Hence the writer is of the opinion that, other conditions being equal, that design is most desirable which provides the maximum number of closest spacing of holes, regardless of diameter.

*Quantities.*—How much and what kind of grout shall be injected? Cement usually constitutes the second most costly item in a grouting program. The writer is in complete agreement with Mr. Lewis' conclusions that:

> (*a*) "In the future, the major economies that will be effected in the field of foundation treatment will probably result from the development of cheaper materials for grouting" (see heading "Conclusion").
>
> (*b*) "* * * it [is] unwise to use sand for the primary grouting of seams in a foundation that will be subjected to more than a moderate head" (see heading "Use of Rock Flour: Sand for Grouting").

Among the emergencies which arise and must be coped with by the field forces without the delays incidental to consultations and special studies, as suggested by the author under the heading "Conclusion," is the case of the "unusual" hole that consumes an exorbitant amount of grout. It is true that the quantity of grout that can be injected into a hole is very materially reduced by using a sanded mix or by pumping thick grout slowly, but such procedure is thought to be a form of self-deception. The grout consumed is "exorbitant" because a weakness has been disclosed by the grouting which exploratory drilling failed to expose. Using a sanded mix or pumping thick grout slowly merely masks what may be a highly undesirable condition. Filling the reservoir can well be associated with excessive hydrostatic uplift pressures and leakage through the dam abutments. Regrouting then becomes a necessity. The writer has had occasion to so regrout three major structures and hence looks with extreme suspicion upon attempted economies in the quantity of grout that a hole will consume.

*Methods.*—It should be borne in mind that the job will be made safe in the field and not in the office. There is perhaps no phase of engineering or construction which does not lend itself more readily to rule-of-thumb or handbook methods than does foundation grouting. Exhaustive preliminary studies, competent design, stringent specifications, and excellent equipment lose much of their value when the actual work is done by low-grade labor under mediocre supervision. The field methods used are of vital importance. How much dependence can be placed in so-called washing operations? Should screened cement be required? Should stage grouting be used? Space does not permit giving much time to these important questions. It is believed that seam washing should be done as thoroughly as the author describes, or it should not be attempted. The writer has had occasion to expose, by excavation, seams which had been incompletely washed before grouting and found the contained grout to be a putty-like paste, the clay having intermingled with the grout to an extent which prevented the cement from taking a set.

Should rescreened cement be used? Fineness is undoubtedly desirable in grouting the fine seams in foundation rock. However, rescreening costs are relatively high, and it is possible to remove oversized particles by using the grout delivery line as a wet classifier. This method takes advantage of the sorting action of flowing water carrying suspended matter. No deposition takes place as long as a hole consumes grout freely, but as it tightens up the velocity of the flowing grout decreases, thereby depositing the oversize particles in the invert of the pipe line from where they are readily removed by occasionally sluicing out the line.

Should stage grouting be used? Although this is a more costly method than full-depth grouting, the additional cost is thought to be warranted in many cases. The openness of seams in rock ordinarily decreases with depth, and consequently the grout consumed per foot of hole is greater near the surface where shrinkage is greatest. Grouting in sections from the bottom of the hole upward, by means of packers, permits the use of higher pressures in the lower reaches and eliminates the necessity for cleaning the hole after each grouting. However, it does not subject the near-surface rock to the repeated groutings of the so-called "stage" method. It is the successive introduction of grout into this critical zone which compensates for shrinkage and holds to a minimum the tendency for grout to return from "holes that had been previously grouted to refusal" (see heading, "Shallow Grouting").

C. E. Blee,[29] M. Am. Soc. C. E. (by letter).[29a]—All of the papers presented in this Symposium are full of interest in that they give the geological conditions encountered and the foundation treatment developed to meet these conditions. The writer has been particularly interested in the foundation treatment at Chickamauga Dam, having had the opportunity of visiting this work from time to time while it was in progress.

The grouting used as a construction measure to tighten the cofferdams and to reduce the flow of water into the cutoff trench excavation might be regarded as of secondary importance, but actually was a major operation and quite vital to the completion of the entire project. The development of what Mr. Hays has termed "prescription" grout, consisting of sand, bentonite, and cement, is of particular interest. Where the quantities involved reach such large amounts as they did at Chickamauga Dam (with one item of "prescription" grout exceeding 150,000 cu ft), it seems almost an economic necessity to develop a low-cost grouting material. Along the same line is the development of the stabilized clay grout, formed by the addition of portland cement to a local clay, used to fill cavities outside of the cutoff but under the earth embankment.

The grouting under the concrete portions of the dam was more along conventional lines. It consisted of consolidation grouting and cutoff grouting similar to that which might be encountered in any concrete dam but more extensive than is usually found.

In reviewing the procedure used for obtaining the cutoff in the earth-dam sections, one is impressed by the thoroughness, yet flexibility, of the methods used. The 36-in. core drill, although introduced primarily as a means of exploration, has developed into a very useful construction device for cutting a shaft to give access to underground work. At Hiwassee Dam, in one particular area of the foundation, there was a comparatively narrow seam of disintegrated rock overlaid by about 20 ft of sound rock. In order to treat this condition without removing the overlying sound rock, a number of 36-in. holes were drilled in this particular area. Workmen were then able to go in and, operating from these holes by mining methods, successfully cleaned out the seam of disintegrated rock, which was then backfilled with concrete and later pressure grouted.

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[29] Project Mgr., Fort Loudoun Dam, TVA, Lenoir City, Tenn.

[29a] Received by the Secretary June 18, 1940.

After reading the paper by Mr. Hays, it appears that there should be no question as to the adequacy of the cutoff obtained in the earth-dam sections. Rather, there might be a question as to whether a less extensive treatment would have sufficed. With respect to engineering structures in general, and dams in particular, this is a question that is seldom answered even by experience with the structure in use. In the case in question, it is evident that steel sheet piling could not be used on account of the mass of boulders and rock fragments overlying bedrock. The open trench excavation was the logical method of reaching bedrock and had the further advantage of affording a means of excavating through the upper and more disintegrated portions of the limestone. Mr. Hays states that such excavation, together with the resulting backfill, was found to be more economical than the excessive grouting that would have been required to fill the openings in this surface rock.

The adopted spacing of 12 in., center to center, for the final grout holes in the cutoff may seem rather on the conservative side. It must be recognized, however, that the cost of these holes, driven with wagon drills, is not a large item and if half the holes had been omitted, no enormous saving would have been realized. In looking at the cutoff on the site, particularly near the outer limits of the south embankment, one was apt to have the impression that with so much overburden a long leakage path would be presented which would materially aid in accomplishing the cutoff. Apparently, however, from the large amount of pumping required during construction, open water passages extended to the outcropping of the rock in the river channel, and the cutoff had to withstand the full hydrostatic head of the reservoir.

The writer agrees that, in constructing a cutoff by grouting, it is advisable to place the holes all in one row with "the pickets of the fence" close together rather than to use staggered rows.

The paper by Mr. Hays describes one of the most extensive projects ever to be undertaken in foundation treatment for a dam. It is particularly valuable in showing the possibility of making available dam sites which might otherwise have been considered impracticable.

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# DISCUSSIONS

# FLOOD-PROTECTION DATA

## PROGRESS REPORT OF THE COMMITTEE

### Discussion

BY O. J. TODD, M. AM. SOC. C. E.

O. J. TODD,[9] M. AM. SOC. C. E. (by letter).[9a]—The Progress Report of the Committee on Flood-Protection Data is particularly valuable in that it emphasizes the need for greater care in arriving at conclusions in dealing with various aspects of complex flood problems.

Under "III. Statistical Methods" the Committee properly frowns on the unwise use of the flood-frequency method for computing what is called the "average annual flood damage." A fixed stage-damage relation is difficult to maintain, especially where trees and growing crops are concerned, unless the same or similar plant life remains from year to year on the terrain considered.

The floods along the Potomac River in Washington, D. C., quickly ruined the beautiful grove of flowering cherry trees from Japan a few years ago. Weeping willows would have survived this inundation. When floods covered 10,000 sq miles of agricultural land near Peking and Tientsin, China, in the summer of 1924, corn crops were soon ruined, whereas kaoliang (tall stalks of similar character in many respects) thrived. The following year the hazard might be less and the stage-damage relation greatly altered with the planting of more kaoliang. In fact, agricultural populations are constantly seeking to insure themselves by varying their planting to conform to flood hazards.

"VI. Cooperation Between Agencies" in compilation of hydrologic data receives special comment from the Committee because of the fine spirit that has been promoted in recent years between federal bureaus and with other organizations. This is especially true in the United States. It has been much less so in parts of the Orient. Too much emphasis cannot be placed on the need for a thorough coordination of hydrological and meteorological agencies in the problems of flood control. Pioneer efforts by the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., in 1919, in connection

NOTE.—This Progress Report of the Committee on Flood-Protection Data was published in April, 1940, *Proceedings*. Discussion on this Report has appeared in *Proceedings*, as follows: June, 1940, by Waldo E. Smith, M. Am. Soc. C. E.

[9] Engr., Bureau of Reclamation, Denver, Colo.

[9a] Received by the Secretary July 29, 1940.

with Grand Canal studies and Yellow River flood problems, called attention to this need. Until that time provincial and national agencies had gathered very little dependable data useful to engineers working on the Yellow River and other streams of North China, although the Chinese Maritime Customs service had records of river stages dating back over half a century on the navigable Yangtze River. These had been gathered under foreign supervision for the most part. China had no weather bureau. Engineers have depended on Sicawai Observatory in Shanghai (operated and owned by the Jesuits) for dependable rainfall data, aside from scattered records in other port cities where foreign influence had prevailed. The vast interior of China was without rainfall or runoff data of any consequence.

Following Mr. Freeman's work of 1918 and 1919, other members of the Society have endeavored to secure a more extensive distribution of rain gages, but with only partial success. Most of the native observers were badly paid, with the expected results. Rainfall records from scattered mission stations, agricultural schools, and magistrate's yamens (county offices) were too often undependable.

Therefore, it fell to the lot of special engineering commissions, such as the Chihli River Commission (1918–1937) and later the Yellow River Commission (1933–1937), to gather data in a systematic manner. All foreign engineers engaged in river control or irrigation in China helped in this movement, which received support from the Chinese Government, so that the number of dependable stations grew annually until the summer of 1937 when war brought to an end much of this work. The chaos caused by war will produce a serious gap in these records, which gap will remain until peace is reestablished.

Another item included in the Committee's Report is that of "VII. Floods Caused by Ice." In China, as well as America, this aspect of flood control has been somewhat neglected. However, for many decades the Yellow River Bureau of Shantung followed out a program of protection of revetment work against damage by ice. The willow poles 4 in. to 6 in. in diameter and 16 ft to 20 ft long are grown along the inner toe of the dikes. In late autumn these poles are roped or wired in vertical position on the faces of kaoliang groins or revetment, especially where currents are bearing in on banks.

At times heavy ice jams on the upper Yellow River give way early in the year sending many miles of floating ice chunks downstream. These may catch and form new jams at critical bends or at bottlenecks in Shantung Province. The writer has witnessed two serious jams of this nature within 100 miles of the river's mouth. In both cases the jam raised the water immediately above to such heights that main dikes were breached and heavy damage to inundated farm lands resulted. In one instance the jam came at a bottleneck in a fairly straight course. A realinement of dikes and removal of the bottleneck would prevent a recurrence of this trouble. Additional height and section for dikes in regions where ice jams are to be expected should be provided.

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DISCUSSIONS

# THE PRACTICE OF STATE HIGHWAY DEPARTMENTS IN THE DESIGN OF ABUTMENTS

## PROGRESS REPORT OF A SPECIAL SUBCOMMITTEE OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON MASONRY AND REINFORCED CONCRETE

### Discussion

BY J. WAYNE COURTER, ASSOC. M. AM. SOC. C. E.

J. WAYNE COURTER,[2] Assoc. M. Am. Soc. C. E. (by letter).[2a]—The writer has read with great interest the Committee Report on the practice of the several state highway departments on the design of abutments. The Report shows that there is considerable variation in ideas on both design and type.

Although the design assumptions are fairly well fixed by the Bridge Specifications, American Association of State Highway Officials (AASHO) (which are believed to be in general use), it would seem that these specifications are altogether ignored by those states which proportion spill-through abutments by individual judgment rather than by actual analysis of some kind. Although the aforementioned specifications do not permit consideration of the overturning resistance offered by earth in front of an abutment, some engineers assume unbalanced earth pressure only on that part of the structure above the natural ground line.

Regarding overturning of spill-through abutments, one case comes to mind in which the maintenance forces, because of frequent washouts of the fill, boarded up the openings between the columns, thus producing a full-retaining type. That the abutment suffered no ill effects from the apparently increased overturning moment would seem to bear out Mr. Overholt's contention that there was no such increase (see heading "Design").

The usual argument in favor of spill-through abutment construction is that the cost, together with the additional length of bridge necessary to provide equal waterway area, is less than the cost of full-retaining abutments. This

NOTE.—This Report was published in June, 1940, *Proceedings*.

[2] Associate Highway Bridge Engr., Public Roads Administration, Denver, Colo.

[2a] Received by the Secretary July 18, 1940.

is undoubtedly true, and there are cases where the increased maintenance expense of the former type has not offset the saving in first cost. Nevertheless, the writer favors the use of spill-through abutments only at those places where water action is a minor factor, such as grade separations and canal or small lake crossings.

Mr. Ornburn (heading "Types") favors the open construction because the cost of replacing a washed out fill is negligible when compared to the cost of replacing a structure. This is unquestionably true, but the writer believes that bridges should be sufficiently large to care for floods without any washouts. On an important highway, the cost of delays to transportation due to a washout may be considerable. Also, the good will of the traveling public is likely to be washed out along with the fills. If limited funds make it necessary to construct a bridge of insufficient length for maximum flood, a spillway or overflow section can easily be built in the approach fills to protect the structure from washout regardless of type of abutment used. In any case, a bridge of this kind should have abutments so designed that they may be easily converted into piers and the bridge extended to the proper length when future funds become available.

If spill-through abutments must be used, the fills around them should have flat, sodded slopes, not steeper than 1 on 2. The fills should be placed in thin horizontal layers, watered, and thoroughly compacted for maximum density. Due consideration should be given to the type of fill material used. Where water action is an item of importance, the writer suggests the use of a fill section as shown in Fig. 1. The thin concrete or rock slope paving that is

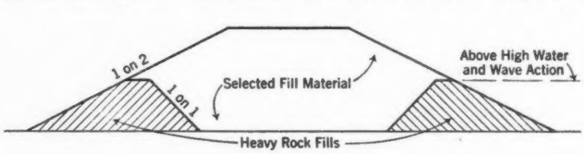


FIG. 1.

sometimes used below water line is not believed to be very satisfactory under extreme flood action.

Settlement of fills behind abutments is not always due to improper placing of the fill itself, but rather to the inability of the natural ground to support the fill adequately. The foundation for a high fill is just as important from an engineering standpoint as that for a structure, but this fact is not always recognized. Unsuitable material should be removed and the fill built up from a firm foundation which will not compress more under its load than does the material under the abutment footings. If this condition is achieved, and the fill properly constructed of suitable material, there should be no bump at a bridge end.

The writer wishes to thank the Committee for publicizing the results of its questionnaire, and he feels that much good will result from a study of the Report and the subsequent discussions.

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DISCUSSIONS

# RECOMMENDED PRACTICE AND STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

## Discussion

By L. J. Mensch, M. Am. Soc. C. E.

L. J. Mensch,[6] M. Am. Soc. C. E. (by letter).[6a]—This important document should be the last word on the present state of the art of reinforced concrete. In some instances this is not the case, and the writer wishes to point out a few of them.

Chapter III, on "Proportioning, Mixing, Curing, and Testing Concrete," contains what seem to be most surprising innovations in the art, purporting to reflect the progress made since 1924. First, the statement is made that there are two ways of controlling the strength of the concrete in a structure. For example, in alternate A (paragraph 304) only the strength and the water content are specified, and the responsibility for the strength and other good properties is placed upon the contractor; in alternate B (paragraph 305) the engineer is assumed to know his business and tells the contractor what to do.

Then the engineer is offered, for use in alternate A, the values in Table 1, which would make any young engineer or an inexperienced contractor believe that the water content is the most important constituent of concrete. In this the Committee disregards the teachings of such eminent pioneers as Bauschinger, Bach, Durand-Claye, and especially R. Feret. For 50 years the latter has been (and still is) investigating this particular subject; and he maintains that the strength and durability of the concrete depend:

(1) Primarily on the cement factor and the quality of the cement;

(2) Secondly (and very often nearly as important as the cement), on the granulometric and mineralogical composition of the sand, especially when the particular deposit does not have a successful record of several years' use in concrete work, or in case of marine structures and alkali soils;

Note.—This Report was published in June, 1940, *Proceedings*, Part 2.

[6] Civ. Engr. and Constructor, Chicago, Ill.

[6a] Received by the Secretary July 5, 1940.

(3) On the granulometric composition of the coarse aggregate, its hardness, density, water absorption, the existence of clay seams, etc.

(4) On the workmanship in mixing, placing, and curing the concrete; and

(5) The water-cement ratio.

For 50 years and more the influence of the water-cement ratio was slight, as low water-cement ratios were always specified. Originally, heavy tamping was required; and later, with the introduction of reinforced concrete, quaky mixtures came into use, requiring spading and less heavy tamping. The water content was formerly given in percentages of the weight of the dry mixtures, varying from about 6% to 8%; but when consulting engineers and architects began to give concrete work to the carpenter instead of to engineering contractors, very wet concrete came into fashion, considerable trouble with laitance appeared, and frost damage in northern climates became serious. It is to the credit of D. A. Abrams,[7] M. Am. Soc. C. E., that he has reminded American engineers that the water-cement ratio must be low in order to obtain durable concrete; and he expressed it as the ratio of volume of water to that of cement. The Committee expresses the water content in gallons per sack of cement.

The writer claims that to specify only the strength of the concrete and the water-cement ratio will give rise to many disputes and also to poor structures. For example, suppose that a water-cement ratio of 5.5 gal is specified for a sea-wall. If the contractor works under alternate A, neither he nor the supervising engineer would know offhand what mixture of concrete is desired. By consulting Table 2, however, they can find that 4,500 lb concrete is required. Now the contractor, being supposedly skilled in his business, will most likely proceed as follows: He will use a mixture of 1 bag of cement to $2\frac{1}{2}$ parts of sand, measured by damp and loose volumes, and 4 parts of 2-in. gravel, with the specified $5\frac{1}{2}$ gal of water, and he will obtain approximately a 1-in. slump. By vibration he may easily produce a 5,000-lb concrete, but it will not make a durable sea-wall when exposed to salt water, waves, and frost.

A richer mixture containing at least 6 to $6\frac{1}{2}$ bags of cement (instead of about $5\frac{1}{4}$ bags in the foregoing mixture) is required, and the sand must be selected most carefully. No warning in this respect is given in Table 1. Water-cement ratio seems to be the cure-all. Another contractor may look at Table 3 and may believe that it requires more than 8 sacks of cement in a yard of concrete in order to obtain 4,500 lb concrete with a water content of $5\frac{1}{2}$ gal per sack of cement. Therefore, he will prepare his estimate on this basis and will probably be an unsuccessful bidder. It leads to confusion. Nothing is said in Table 1 about the temperature of concrete and air; it is well known that it often takes 20% more water in concrete operations at 70° F than at 45° F and still more at 80° and 90°. A water content of $5\frac{1}{2}$ gal may be excellent for a slab in a building, but may lead to surface scaling in a road slab. It will lead to the formation of laitance and weak concrete in a pier 5 ft square and 20 ft high, all for the same cylinder strength of concrete (say, 4,000 lb per sq in) and the same slump.

[7] "Design of Concrete Mixtures," *Bulletin No. 1*, Lewis Inst., Chicago, Ill., 1919.

In the aforementioned pier the concrete in the lower part, even when the slump is 1 in. or 2 in., will give up water to the higher part during concreting operations, and finally the top layer, from 1 in. to 6 in. thick, may be full of laitance, and may be a very weak link in the structure. This water gain is experienced in dam construction when blocks are only from 2 ft to 5 ft deep.

M. Considère experienced[8] this water gain in his test columns, and in order to obtain more reliable results used drier mixtures in the top part of the columns; competent engineers have specified such variations of water content for many years, but the Committee has failed to give cognizance to it in constructing Table 1.

It is the writer's opinion that textbook and specification writers are making a "mountain out of a molehill" in their treatment of concrete proportioning. The matter is really very simple. From a large number of tests which Professor Abrams sent to the writer a number of years ago, the writer is able to offer the following rules for the design of concrete mixtures, provided that either the Feret[9] or Abrams[7] method is used for selecting the grading of the fine and coarse aggregate.

The water content of a concrete mixture depends upon three factors:

(1) The cement content: When $23\frac{1}{2}\%$ of water by weight is added to most cements, the densest mixture of cement and water is obtained, which experience shows is also the strongest mixture. Therefore, the assumption is made that for each sack of cement it is necessary to use $0.235 \times 94 = 22$ lbs of water, or 2.65 gal, whatever the slump.

(2) To make the cement-sand-water mixture of the desired workability and strength, a certain percentage of water must be added to the weight of the sand, which percentage depends on the grading of the sand and the slump desired. On the average, the percentages are as shown in Col. 3, Table 7.

TABLE 7.—WATER REQUIREMENTS OF CONCRETE

| Slumps (in inches) between: | Percentage of Water for | | | Twenty-eight day compressive strengths (in lb per sq in.), per sack of cement |
|---|---|---|---|---|
| | Cement | Sand | 1-in. Gravel | |
| (1) | (2) | (3) | (4) | (5) |
| 1 and 2 | $23\frac{1}{2}$ | 6 | 3 | 700 |
| 3 and 4 | $23\frac{1}{2}$ | $6\frac{1}{2}$ | $3\frac{1}{4}$ | 650 to 675 |
| 5 and 6 | $23\frac{1}{2}$ | 7 | $3\frac{1}{2}$ | 600 to 625 |
| 7 and 8 | $23\frac{1}{2}$ | 9 | $3\frac{1}{2}$ | 550 |

(3) To make the cement-sand-coarse aggregate-water mixture of the desired workability, a certain amount of water must be added to the coarse aggregate which, in percentages of the weight of the coarse aggregate, is given in Col. 4, Table 7.

This particular series of tests also showed what 28-day strengths of concrete, with standard curing, may be obtained for each 94 lb of cement of the

[8] "Expériences, Rapports, Instructions," Commission du Béton Arme, Paris, 1907.

[9] Cement and Concrete, by L. C. Sabin, McGraw Publishing Co., 1907, p. 205.

latest standard type contained in a cubic yard of concrete, for the various slumps, as shown in Col. 5, Table 7. Of course, all values are approximate and the strengths are valid for cement factors of 5 to 8 sacks per cu yd. For a cement factor of 4, the strengths should be reduced 20%, and for a cement factor of 10 they should be reduced 10%.

These tests further showed that the foregoing differences of strength of 28-day concrete are considerably reduced at three months and one year. For example, consider a mixture of 1 bag of cement, 200 lb of dry sand, and 396 lb of dry, 1-in. gravel which corresponds to a mixture, by loose damp volumes, of about 1 : 2.25 : 4. A slump of 1 in. to 2 in. will require a weight of water per sack of cement equal to 22 lb (for the cement alone) $+ \frac{6}{100} \times 200$ (for the sand alone) $+ \frac{3}{100} \times 396$ (for gravel alone) = 45.88 lb = 5.5 gal. This mixture requires a cement factor of about 5.6 sacks (may be computed also by the methods of absolute volumes) and should show a 28-day strength of about $5.6 \times 700 \doteq 3{,}920$ lb per sq in. Professor Abrams found 3,950 lb per sq in. Excellent agreements with this rule were found for thousands of tests.

The writer has only a few tests on concrete mixtures with 2-in. aggregates, but these indicate that for the same strength, slump, and workability there is a saving of about 10% in cement over mixtures containing 1-in. aggregate and that there is slightly less water needed.

In foundation walls, footings, and thick floor slabs, 2-in. aggregate has been used with great success, which is in contrast with suggestions in Table 4, paragraph 307.

The water content needed to obtain workable concrete is quite different when crushed stone sand is used. Practically the only worth-while tests were those reported by Mr. Feret.[9] With proper grading he showed that such sand may give more superior results than natural sands of similar good grading. On the other hand, many stone sands are graded improperly and contain disintegrated particles that produce inferior results. The enthusiasts for the water-cement ratio state that the reason is that more water is required; but the inferior result is obtained likewise with a very low water-cement ratio. The water content must be changed also when crushed stone instead of gravel is used. It depends also on the kind of stone.

The most serious "enemy" of good concrete in many types of reinforced-concrete structures is the water gain by which voids beneath the longitudinal steel rods in beams weaken the bond and the resistance to compression. Weak concrete forms at the tops of beams where strength is needed in compression; and joint surfaces, walls, and columns at the top are likewise weakened as previously mentioned.

To summarize the writer's criticism of Table 1: No one but an "expert" in proportioning by the water-cement ratio would understand what this table means, or what concrete mixtures are required. The Report does not instruct the user as to how this table must be changed when concrete is poured at different temperatures from, say, 45° to 100° F. Neither are instructions given as to how to prevent water gain in various concrete members. No cylinder

tests would ever reveal the damage done by water gain. Neither are instructions given as to how to change the water content for the various gradings and qualities of the aggregate. In many cases, a 20% increase in the water-cement ratio may give better durability and imperviousness and also higher strength than the ratios suggested in the table.

In Chapter VIII, on design, no improvements are to be found in paragraph 802. In the discussion[10] on the first Committee Report, the writer stated that such assumptions will hinder the clear understanding of the mechanics of reinforced concrete for another ten years or more. In view of the present Report he confesses that his former guess was a great error. He should have estimated the time at fifty years rather than ten years. Furthermore, he stated that formulas based on the conditions prevailing at the ultimate load were considerably simpler and also more correct, because they agreed with facts.

Doubtless, the pioneer engineering contractors never used this theory, which was originally conceived by E. Coignet and de Tedesco[11] and adopted by the French Commission of Reinforced Concrete in its Report.[8] M. Considère and Maurice Levy gave reasons why these assumptions were adopted by the French Commission, although they were well aware that the assumptions did not apply to a heterogeneous material such as reinforced concrete. At that time there were so many systems of reinforced-concrete design, promoted by high-pressure salesmanship with fanciful design methods, that the Commission considered it best to rely on a principle well known to every educated engineer —namely, the classical theory of elasticity. In order to make the resulting designs more in agreement with current practice of the most successful engineering contractors, they allowed design stresses which were far in advance of anything permitted by building regulation at that time, and the modular ratio, $n = \frac{E_s}{E_c}$ was made variable from 8 to 15. The result was satisfactory for the current practice of that time. This theory was adopted by code authorities all over the world, but the new code makers did not notice that the theory gave economical results only when the working stresses were in conformity with those adopted by the French Committee. The new codes used much lower stresses, leading to uneconomical results, which explains the many changes of building codes that have occurred in the United States.

Since nearly all textbooks teach only this theory, the writer will give four reasons why it should be thrown into discard:

(1) The straight-line deformation of a cross section under flexure is only approximately true in the uncracked portion of a member. Observations have shown that there are visible (and also invisible) cracks in beams when the steel stress reaches a value of 4,500 to 6,000 lb per sq in. In 1905, the writer described[12] extensometer tests on beams with artificial cracks produced by vertical greased zinc plates. The measurements were taken on a 2-in. gage length across the crack at various heights from the bottom, and very serious deviations from a straight-line deformation were observed.

[10] *Transactions*, Am. Soc. C. E., Vol. LXXXII, December, 1918, p. 1541.
[11] *Mémoires* de la Société des Ingénieurs Civils de France, 1894.
[12] *Journal*, Western Soc. of Engrs., 1905, p. 355.

(2) The modulus of elasticity is constant neither for concrete in compression nor for concrete in tension (which becomes zero at a very low load) nor for steel after a strain of about 0.001 in. per in. is reached. The modulus of concrete in compression varies with the stress and with time; and the so-called modular ratio has been evaluated by different investigators at from 6 to 100, from their own observations.

(3) The assumption that the adhesion of concrete to steel is perfect within working stresses is directly contradicted by the observations of Professor Abrams,[13] who described serious movements in longitudinal bars between concrete and steel in the middle of reinforced-concrete beams at steel stresses of only 6,000 lb per sq in.

(4) It is well known that Hooke's law does not apply to a plain concrete beam.

By manipulating the working stresses the French Commission obtained a good agreement with practice, and finally after thirty years this Committee also manipulated the working stresses and the so-called modular ratio so that for low percentages of reinforcement of one grade of steel (intermediate grade) a fairly uniform factor of safety may be obtained by the use of the complicated formulas which are the results of the design assumptions of paragraph 802.

For some unaccountable reasons the practice has been developed in the United States to favor one particular percentage of reinforcement in slab or beam design. This is the so-called balanced reinforcement, which is not balanced in nature, and varies with every code.

The Joint Report permits the working stresses for concrete, $f_c = 0.45 f_c'$ and $f_s = 20{,}000$ lb per sq in. for intermediate grade of steel, with the result (not mentioned by the Committee) that, for all strengths of concrete, the balanced steel ratio is given by the relation

$$p = \frac{A}{b\,d} = \frac{4.5 f_c'}{10^6} \qquad (19)$$

and the permissible internal resisting moment by

$$M = 0.0787\, b\, d^2 f_c' \qquad (20)$$

Hence, for 3,000-lb concrete, $p = 0.0135$ and

$$M = 236\, b\, d^2 \qquad (21)$$

The permissible stress of $0.45 f_c'$ will be considered unreasonably high by most engineers and will make them believe that designs according to this recommended practice imply a factor of safety of 2.2 in the most favorable case, under laboratory conditions. The yield point of intermediate grade steel is guaranteed only to 40,000 lb per sq in., which will further confirm them in the belief that the Committee's recommendations in fact mean that a factor of safety of 2.0 is sufficient for good practice. As stated previously, however, the design assumptions are wrong, and tests[14] show that the factor of safety is between 2.5 and 2.6.

[13] *Bulletin No. 71*, Univ. of Illinois, 1913, p. 216.

[14] *Journal*, Am. Concrete Inst., September–October, 1936.

It is no rarity with the average contractor to produce 2,000-lb concrete instead of the specified 3,000-lb concrete. According to these design assumptions, a slab of 2,000-lb concrete with 1.35% of intermediate grade of steel would be good for a permissible bending moment of only 177 $b\ d^2$, or 25% less than the aforementioned 236 $b\ d^2$ permissible for 3,000-lb concrete. However, tests to destruction indicate an ultimate moment for slabs of 2,000-lb concrete and 1.35% of reinforcements of 545 $b\ d^2$, resulting in a factor of safety of 545 : 236 = 2.31 instead of 2.55 for 3,000-lb concrete, or only 9% less, instead of the aforementioned 25%.

In beams continuous at the support the engineer often encounters much higher percentages of steel than the so-called balanced reinforcement; 3% and 4% are not unusual. In case of 2.5%, these design assumptions, when using 3,000-lb concrete, result in: $k = 0.50$; $j = 0.833$; and

$$M = \frac{0.50}{2} \times 1{,}350 \times 0.833\ b\ d^2 = 281\ b\ d^2 \qquad \text{(22)}$$

whereas tests reported by Inge Lyse,[15] M. Am. Soc. C. E., indicate an ultimate moment of 960 $b\ d^2$, or a factor of safety of $\frac{960}{281} = 3.42$. This is quite different from the factor of safety of about 2.55 for the so-called balanced reinforcement.

When the engineer specifies the high-yield-point type of steel, furthermore, these design assumptions result in still other factors of safety. For the lowest grade of high-yield-point steel in the market, tests by Professor Lyse[15] show that beams with 1.35% of reinforcement fail at an ultimate moment of about 778 $b\ d^2$. The Joint Report does not recommend a higher stress on high-yield steel than on intermediate grade steel; therefore, the permissible value of $M$ is as given by Eq. 21—namely, 236 $b\ d^2$—and the factor of safety in this case becomes $\frac{778}{236} = 3.3$, against 2.55 for intermediate grade steel.

For beams with compressive steel the design assumptions in paragraph 802 result in factors of safety of from 2.55 to 4.6; therefore, these assumptions do not lead to uniform degrees of safety.

Elsewhere the writer has shown[16] how to avoid the pitfalls of the straight-line theory. Thomas Barlow revealed[17] these pitfalls as early as 1814 for other materials like wood, cast iron, etc.; yet the scientific world still insists on looking down at any speculations based on the conditions at the time of failure, as being of inferior merit.

That this Committee itself did not have too much confidence in the classical design assumptions is shown by the recommendations contained in paragraph 802, item 3, and paragraph 804, item (*c*), for compression reinforcement in columns and beams. For beams the arbitrary ruling is made that the stresses in the compression reinforcement may be taken at twice the value resulting from the assumption of the straight-line theory but not to exceed 16,000 lb

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[15] *Journal*, Am. Concrete Inst., September–October, 1936, Test No. 15.

[16] *Loc. cit.*, December, 1914; *Proceedings*, Am. Concrete Inst., 1937, p. 498; and *Transactions*, Am. Soc. C. E., Vol. LXXX, December, 1916, p. 1734, and Vol. LXXXIII, 1919–1920, p. 1667.

[17] "The Strength of Timber," by Thomas Barlow, Research Engr., Woolwich Arsenal, London, 1814.

per sq in. When the location of the compressive reinforcement is more than 0.25 $d$ distant from the compression face of the beam (which easily occurs in continuous beams), then an increase of 200% to 300% of the ideal compression stresses given by the straight-line theory would still result in designs which do not agree with tests. This recommendation indicates to the writer that the straight-line theory had no solid basis in the minds of the Committee.

In paragraph 804, item (*a*) 1, designers are advised to use, as effective span, the center-to-center distance between the supports. This seems like a very conservative recommendation; but the real meaning is that the moment thus obtained by the classical theory of elasticity for continuous beams may be reduced to the moment theoretically possible at the edge of the support, which gives a smaller moment than that obtained if the clear span were used in most cases. Furthermore, it results in a wrong location of the point of contraflexure and may induce engineers to make the negative bars too short over the supports.

The recommendations for the design of two-way slabs with supports on four sides are more rational than those of most building codes; it cannot be said, however, that they are brought to the reader's notice in as clear a manner as the design of flat slabs in paragraphs 835 to 837, or that they are as favorable as the latter recommendations. In the progress report, this Committee[18] stated that it was not yet prepared to make recommendations for this type of slab. At the American Concrete Institute convention in February, 1937, a discussion by the writer was read (not published), in which he advised the Committee to adopt the design formulas of F. Hennebique,[8] the engineer who first promoted the use of such slabs and originated the following empirical formulas:

Let $a$ and $b$ be the sides of a rectangular panel between beams (clear spans); then the average positive moment per linear foot for an inside panel, in either the long or short direction, is given by

$$M = \left(\frac{a + b}{2}\right)^2 \frac{w}{54} \dots\dots\dots\dots (23)$$

and the negative moment over the supports in either the long or short directions by

$$M = \left(\frac{a + b}{2}\right)^2 \frac{w}{36} \dots\dots\dots\dots (24)$$

The writer further advised the Committee to increase these moments by 20% for each outside edge.

Mr. Hennebique originally used the denominator of 36 in Eq. 23, but with experience gained by years of application the denominator 54 was finally adopted about forty years ago.

The Committee's recommendations follow substantially Mr. Hennebique's empirical formulas. For example, his formulas (Eqs. 23 and 24) for average moment have been changed as follows:

Positive moment in the short direction—

$$M = \frac{w}{48}\left(\frac{a + b}{2}\right)^2 \dots\dots\dots\dots (25a)$$

[18] Progress Report, Joint Committee, issued by Am. Concrete Inst., January, 1937.

Negative moment in the short direction—

$$M = \frac{w}{36}\left(\frac{a + b}{2}\right)^2 \qquad (25b)$$

Positive moment in the long direction—

$$M = \frac{w\,S^2}{48} \qquad (25c)$$

Negative moment in the long direction—

$$M = \frac{w\,S^2}{36} \qquad (25d)$$

in which $S$ refers to the short span regardless of the value of the long span, and all spans are measured from center to center of beams, instead of Hennebique's clear spans. For square panels, Table 5 gives the positive moment for the middle strip as $\frac{w\,S^2}{40}$, and paragraph 811 advises the designer to make the moments in the side strips $\frac{w\,S^2}{40} \times \frac{2}{3}$ (all per linear foot). By adding these two values and dividing by two the average moment is found to be $\frac{w\,S^2}{48}$, as called for by Eq. 25$a$. Similarly, for the negative moment, $\frac{1}{2}\left\{\frac{w\,S^2}{30} + \frac{w\,S^2}{30} \times \frac{2}{3}\right\} = \frac{w\,S^2}{36}$, as called for by Eq. 25$b$.

Eqs. 25$a$ and 25$b$ apply also to rectangular panels, and the moments for the case, $m = 0.6$, or $b = 1.67\,a$ (or, in the nomenclature of the Report, $b = 1.67\,S$), may be computed as follows: From Eq. 25$a$, the average positive moment in the short direction $M = \frac{w}{48}\left(\frac{1.67\,S + S}{2}\right)^2 = 1.78\,\frac{w\,S^2}{48}$, or 1.78 times as great as for a square panel. Therefore, the coefficient in Table 5 should be $1.78 \times 0.025 = 0.045$. Actually, under heading 0.6, the value given is 0.047, an insignificant difference.

By comparing the coefficients give in Table 5 for Cases 2 to 5 with the coefficients given in Case 1, it will be found they have been obtained from the latter by multiplication with 1.2, $1.2^2$, $1.2^3$, and $1.2^4$, as recommended by the writer for the use of Hennebique's formulas.

The Report made a bold change in the case of the moments in the long direction. The Committee has surmised that both the negative and positive moments are not larger than for a square panel of a length equal to the short side. By making another check of the excellent report by the late W. A. Slater,[19] M. Am. Soc. C. E., the writer finds that the negative moments for rectangular slabs (with $\frac{b}{a}$ varying from 1 to 1.5) agree better with Hennebique's formula (Eq. 24) than with Eq. 25$d$ used by the Committee. For the positive

[19] "Test of a Hollow Tile and Concrete Floor Slab Reinforced in Two Directions," *Technologic Paper No. 220*, Bureau of Standards, 1922.

moment in the long direction, Eq. 25*c* appears to be a better approximation than Eq. 23 in a few cases. No formula has been given for the deflection of this type of slab.

Paragraphs 816 to 830 on diagonal tension, bond, and anchorage could have been written forty years ago. They contain highly dangerous design recommendations and a great many unnecessary restrictions. The Committee needed the results of a well-conceived series of tests of normal and continuous beams with longitudinal reinforcement, for example, as follows:

(1) Longitudinal bars only, the bars to have plain ends;

(2) Longitudinal bars only, the bars to have various types of hooks;

(3) Longitudinal bars only, with plain ends, and with stirrups of various types;

(4) Longitudinal bars only, and with various types of hooks and stirrups;

(5) Longitudinal bars with hooks, one half of the bars being trussed;

(6) Longitudinal bars with stirrups, one half of the bars being trussed;

(7) Longitudinal bars, one half of the bars being trussed and hooked;

(8) Longitudinal bars, one half of the bars being straight, with hooks, and the remainder being trussed and with plain ends.

(9) Longitudinal bars with stirrups, one half of the bars being trussed and hooked;

(10) Longitudinal bars with hooks, and one half of the bars trussed and bent up in various layers;

(11) Longitudinal bars with hooks and stirrups, one half of the bars being trussed and bent up in various layers.

Such a series of tests should give a proper answer to this all-absorbing problem. Actually, such tests have been made, in one laboratory alone, by men of international reputation—about 300 beams, 8 in. by 16 in. in size and 14 ft 6 in. long—and the results have been presented[20] excellently with photographs of all the beams showing the cracks, marked with the load at which they occurred. Similar tests had been made previously by the Austrian Concrete Committee (F. von Emperger, reporter) and by the French Committee, and somewhat later by Professor Abrams[21] and other American investigators, especially the tests by F. E. Richart,[22] M. Am. Soc. C. E.

It was very fortunate that most of these tests were made with 3,000-lb concrete, so that direct comparison is easy. From these tests the Committee could have drawn the following conclusions concerning normal beams with longitudinal bars, which conclusions, in most cases, are in direct contradiction with its recommendations:

(1) Beams with straight bars, without hooks or stirrups, fail at a unit shear of 150 to 180 lb per sq in.

(2) Beams with stirrups and with straight bars, and without hooks, will fail at a unit shear of 180 to 250 lb per sq in. If the stresses of the stirrups in the test specimens, at the ultimate load, are computed by the rules offered by

[20] Publications of the German Committee, Heft 9, 10, 12, 20, 48, and others, 1911, 1912, and 1921.

[21] *Bulletin No. 71*, Univ. of Illinois, Urbana, Ill., 1913.

[22] *Bulletin No. 166* and *Bulletin No. 175*, Univ. of Illinois, Urbana, Ill., 1927 and 1928.

the Joint Committee, they are 20,000 lb per sq in. for stirrups $\frac{3}{8}$ in. and $\frac{1}{2}$ in. in size and 35,000 lb per sq in. for stirrups $\frac{1}{4}$ in. in size. The Committee recommends a working stress of 16,000 lb per sq in. on stirrups of any size and permits a working shear of 180 lb per sq in. on the concrete section, whereas the tests showed a maximum shear of only 250 lb per sq in. The Committee will have a difficult task defending its recommendation.

(3) Beams with straight bars, with hooks, but without stirrups, fail generally at a unit shear of 200 to 250 lb per sq in. Hooks increase the strength of a beam at a rate such that the unit shear is increased from 50 to 70 lb per sq in. The recommendation of the Committee for this case (90 lb per sq in.), although high, is more to the point.

(4) Beams with straight bars, with hooks, and with stirrups, fail at a unit shear of 225 to 350 lb per sq in. The stresses in the stirrups at the ultimate load, when computed according to the Committee's rule, are found to be as given herein under conclusion (2). The recommendations of the Committee could be easily misunderstood by inexperienced engineers, with the result that, in normal beams of this type, they would allow a shear of 360 lb per sq in. on 3,000-lb concrete.

(5) Beams without stirrups but with all bars hooked (right angles or otherwise) and with one half of all bars bent up in one layer, fail at a unit shear of from 250 to 350 lb per sq in. The Committee's rules would allow a shear of only 90 lb per sq in. on most beams of this type.

(6) The bending up of one half of the bars in one layer, even where the angle of inclination was as low as 15° or a trifle less, increased the ultimate unit shear by 50 to 100 lb per sq in. The stresses measured on the inclined portion of the trussed bars in the axis of the beam, whether the inclination was low or approached 45°, were approximately 20,000 lb per sq in. at the ultimate load, whereas the Report allows 16,000 lb per sq in. as a working stress.

(7) The omission of hooks in the straight bars only decreased the ultimate unit shears by about 20%. The same was found for the omission of hooks in the trussed bars, so that the omission of all hooks decreased the strength of a beam by more than 40%.

(8) Normal beams with straight and trussed bars, with hooks, and with stirrups, fail at a unit shear of from 300 to 500 lb per sq in. The stresses in stirrups and in the inclined portion of the trussed bars were found to be as described herein under conclusions (2) and (6). The permissible working stress by the Joint Committee Report would be 360 lb per sq in.

(9) The beams described in conclusion (8), but with the trussed bars bent up in various layers, fail under a unit load of from 350 to 550 lb per sq in.

(10) Beams with welded stirrups are stronger than beams in which $\frac{3}{8}$-in. or $\frac{1}{2}$-in. loose stirrups are used.

(11) Considère hooks (or, as they are termed by the Joint Committee, "standard hooks"—namely, hooks forming a half circle with an inside radius of 3 *d* and with an extension of at least 4 *d*) are not very much more effective, in most cases, than right-angle or acute-angle hooks. They certainly weigh much more and cost more to produce. This extra weight over right angle hooks expended on $\frac{1}{4}$-in. stirrups would produce a stronger beam. Further-

more such bars involve much greater labor cost in handling and placing; they often interfere with other bars in the beams; and they cannot be slipped through the cage of the spiral column reinforcing. The result is that American engineers have developed the habit of omitting hooks both in the straight and trussed bars, with the result that their beams often have rather a low factor of safety.

(12) Beams of very short spans in relation to their depths, or where the point of application is very close to the support, fail at unit shears about 50% larger than normal beams.

In paragraph 823, item (*c*), it is stated that "In order to maintain normal beam action and structural integrity of the member, slipping or appreciable movement of the reinforcement must be prevented." The French Committee's Report,[8] and the report on bond by Professor Abrams,[21] demonstrated the fact that considerable slipping of longitudinal bars has been observed at unit stresses of 6,000 lb per sq in. in the steel at the center of the beam. It is there that the bond is first broken. It is probable that, under working stresses as permitted by this Report, the bars are acting on the concrete only by sliding friction for half their lengths. At conditions nearer to the ultimate load, sliding friction is acting nearly to the ends of the beam, so that the beam may be said to act like a tied arch. With this concept established the importance of end anchorage becomes easily understood. An anchorage of 16 bar diameters as recommended in paragraph 828 is entirely insufficient. This was revealed by the French Commission. Most of the beams on which this discussion was based had an extension, beyond the support, of at least 8 in.; a few had less and others had more.

In paragraphs 851 and 862, treating the subject of reinforced concrete columns, it is stated that, on account of shrinkage and creep, a factor of safety has been adopted varying from 3.6 to 2.75, depending on the amount of vertical reinforcement. On this basis all previous design formulas have been abandoned and brand new formulas have been recommended.

Although it is true that, in some instances, shrinkage and (to a less extent) creep have caused great inconveniences in outside columns of high buildings by cracks in terra cotta and stone covering, no instance has come to the writer's notice in which inside columns caused trouble for the same reasons. Similar difficulties with shortening of outside columns have also been experienced in high, steel-frame buildings due to wind and are counteracted by the use of special joints in the masonry covering. There is certainly no trouble with shrinkage in columns used for reservoirs or in viaducts and similar structures, and engineers should not be required to change their entire mode of thought, and their design practice, without more effective testimony than that presented in the Report.

The tests[23] on reinforced-concrete columns, with time loadings made on behalf of the column investigation committee of the American Concrete Institute in 1932, showed rather high steel stresses and low concrete stresses as found by extensometer measurements in the middle of the columns under

[23] *Journal*, Am. Concrete Inst., January, 1932.

working loads; but at ultimate loads it was found that the strength of tied columns and of bare, spirally reinforced columns was the sum of the strength of the concrete, plus the strength of the vertical reinforced at approximately the yield point, plus the additional strengthening contributed by the spiral reinforcement.

The same conditions affect the problems in flexure. The stresses in steel and concrete, as measured by extensometer tests at the working load, are quite different in quality from those measured near the ultimate load. Still, the Committee did not make any change in the design assumptions in flexure, where they were really necessary for beams with high percentages of reinforcing; but they made a change for column design that led (by its own statement) to changeable factors of safety.

By far the most thorough and the most instructive tests on spirally reinforced-concrete columns were made by the French Commission,[8] and M. Considère, the reporter at that Commission, enunciated the following observations which are as true today as they were in 1907.

(1) The effectiveness of the longitudinal reinforcement depends on the contact of the bars with the platens of the testing machine; bars that are covered with 1 in. to 2 in. of concrete (as was done in most German tests and also in many tests made in the United States) may contribute only a few thousand pounds per square inch of their section to the strength of the column, whereas good contact will cause bars to be stressed more or less up to the yield point.

(2) Tamping and the consistency of the concrete has a very large influence on the strength of a concrete column. Tamping of the lower part, as well as the weight of the overlying concrete, causes water gain higher up, and may decrease the unit strength of the concrete of the columns considerably below the strength of test cubes of the same concrete.

(3) The spiral reinforcement may contribute as much as 2.4 times the strength of a vertical bar of the same weight and quality.

(4) High carbon wire of small section and close pitch is much more effective than mild steel wire and wide spacing of the loops.

(5) The pitch of the wire should not be smaller than one third of the core diameter for low concrete stresses and should be less than one eighth of the diameter for high stresses.

(6) Columns have a permanent set of about 0.0002 in. per in. after being loaded to about one fourth of the ultimate load and exhibit considerable flow at higher loads.

(7) The action of the practical column with spiral hooping and protective shell differs widely from that of a bare spiraled column. The protective shell begins to crack at a unit deformation of 0.0015 in. per in. In most tests the protective shell fell off, and the maximum load was reached at a load that was 1.6 times that at which the plain concrete companion columns failed, regardless of the amount of vertical and spiral reinforcement.

(8) The unit strength of the concrete of the protective shell of a column with spiral reinforcement varies from 20% to 60% of that of a plain concrete companion column. This latter point is very important in view of the recom-

mendation of this Committee which requires the use of a spiral reinforcement of such an amount that its contribution is equal to nearly the full cylinder strength of the protective shell. The Chicago Building Code Committee properly rejected this speculation and adopted as minimum reinforcement the amount of spirals which would contribute half the cylinder strength of the protective shell.

Very few additional fundamental facts have been found by later investigators. The tests by the Austrian Concrete Committee[24] have established the facts that: (1) Long, tied columns exhibit an unexpectedly small diminution of strength; (2) steel shows a very large Poisson's ratio at a unit deformation of 0.0015; and (3) designers can ascribe to the strength of the concrete of the core alone, without the spiral reinforcement, about 85% of the cylinder strength.

The tests[25] by T. H. McKibben, M. Am. Soc. C. E., and A. S. Merrill and by Professor Richart[26] have established the unit deformations in the spirals at various unit deformations of the spiraled column in a vertical direction. Professor Richart has also shown that the bare spiraled column fails at a unit deformation in the spiral wire of about 0.005 in. per in. These two series of tests agree in ascribing to the spiral wire quite definite deformations at a unit deformation of the column in a vertical direction of 0.0015 in. per in., depending on the percentage of spiral reinforcement shown in Table 8.

TABLE 8.—REINFORCING EFFECT OF SPIRAL WIRE AT A VERTICAL UNIT SHORTENING OF 0.0015 IN COLUMNS

| Item | Characteristics | Spiral Reinforcement | | | | |
|---|---|---|---|---|---|---|
| | | 0.5% | 1% | 2% | 2.6% | 4.4% |
| 1 | Unit deformation (in. per in.) | 0.00125 | 0.001 | 0.00075 | 0.00065 | 0.0005 |
| 2 | Stress[a] in wire (lb per sq in.) | 37,500 | 30,000 | 22,500 | 19,500 | 15,000 |
| 3 | Additional unit strength (lb per sq in.) contributed to the core by the spiral[b] | 375 | 600 | 900 | 1,014 | 1,320 |

[a] Modulus of elasticity of steel ($E$) times unit deformation (item 1). [b] Equals twice the stress (item 2) times the ratio of spiral reinforcement.

From all the foregoing tests the following conclusions can be drawn for the strength of the practical column with spiral reinforcement and protective shell: At the outset there are two principal types of columns—($A$) that in which the protective shell contributes less than the spiral reinforcement; and ($B$) that in which the protective shell contributes more than the spiral reinforcement. Both types represent good engineering practice where applicable.

The strength of the columns of type ($B$) is the sum of (1) the strength of the protective shell when its strength is assumed at one half of the cylinder strength, plus (2) the strength of the spiraled core when the unit strength is assumed as 0.85 the cylinder strength, plus (3) the contribution of the spiral to the unit strength of the core area at a vertical unit deformation of the

[24] "Versuche mit Eisenbetonsäulen," Eisenbeton-Ausschuss des österreichischen Ingenieur- und Architekten-Vereins, Heft 3, 1912.

[25] *Proceedings*, Am. Concrete Inst., 1916, p. 200.

[26] *Bulletin No. 190*, Univ. of Illinois, Urbana, Ill., 1929.

column of 0.0015 in. per in. (this contribution varies from 375 to 1,320 lb per sq in. of the core area, as shown in Table 8, depending on the amount of spiral reinforcement), plus (4) the strength of the vertical reinforcement. In the actual column, the latter item (4) is somewhat less than the yield point strength; it may be estimated at 32,000 lb per sq in. for mild and intermediate grade steel and somewhat higher for high yield-point steel—say 40,000 lb per sq in. and possibly more. Insufficient tests have been made to establish this last point. Of course, where fireproofing is required, the designer cannot count on the value of the strength of the protective shell, but he certainly can count on this strength in a reservoir or in other structures where there is no danger from fire.

This type of column will fail at the cracking of the protective shell at a unit deformation of about 0.0015 in. per in. This is confirmed by the tests made on 14 columns, 12 in. to 32 in. in diameter, by the American Concrete Institute column investigating committee, and especially by the splendid series of 97 columns by the Dutch Concrete Committee,[27] N. J. Rentgers, reporter. These were 10 in. square and 5 ft long, with ½% to 2% of spiral reinforcement, with 0.2% to 10% of vertical reinforcement, and with concrete strength of 1,700 to 6,500 lb per sq in.

Comparatively few columns of type (*A*) have been tested. The protective shell will fail first at a load of possibly 60% to 80% of the ultimate load, and then the column will finally fail as a bare spiral column with a much larger vertical deformation than 0.0015 in. per in. The writer has measured this deformation in some columns tested by the National Bureau of Standards as 0.01 in. per in. and Considère found even larger vertical deformation.

The strength of the bare column equals the strength of the spiraled core at a unit strength of 0.85 $f_c'$, which is the contribution of the spiral reinforcement to the strength of the core. It may be assumed to be twice the strength of the vertical bar of the same weight and quality plus the strength of the vertical reinforcement at about the yield point (32,000 lb per sq in. for mild, structural, and intermediate grade steel and 40,000 lb per sq in. for high yield-point steel).

All these strength values are based on test columns made with great care in the laboratory and do not consider any diminution of strength due to time loading; the latter effect alone is said to amount to nearly 20%. Any one who has observed the careless manner in which steel setters will bend over the projecting ends of the vertical bars at the top of the column must wonder how the stresses of the bars are transmitted to the concrete; and it is quite likely that often the contribution of the vertical bars to the strength of the actual column in a structure may be at a rate of only a few thousand pounds per square inch., as Considère found in many test columns. To state that the actual strength of a column in the field is 70% of a similar column made and tested in a laboratory is a high rather than a low estimate. Then the factor of safety of 2.75 claimed by this Report becomes only 1.95 in the field.

Considering, furthermore, that most columns are eccentrically loaded and that very few engineers can afford the time to consider secondary stresses (and will not consider them), the factor of safety may be still less. Eq. 9, paragraph 854, for a maximum allowable load, applies to a spirally reinforced-concrete

[27] *De Ingenieur*, 1933, Nos. 27, 31, 40, and 48.

column with protective shell, and gives credit to only one ratio of spiral reinforcement as given by Eq. 10; no credit is given to a higher or lower percentage of spiral reinforcement, quite in contrast with Considère and the building codes of every political unit in the entire world. It was previously stated herein that these formulas were rejected by the Chicago Building Code Committee in 1937. They were not accepted by the New York Building Code Committee and, in the right of the testimony herein presented, should not be accepted by any intelligent engineer. The statement is made in paragraph 851 that the protective shell and spiral reinforcement cannot act together at the same time. How erroneous this statement is the writer has proved to his own satisfaction from the tests made by most competent observers.[8, 25, 26]

Eqs. 9 and 10 and the permissible stresses in the spiral reinforcement place an unwarranted restriction on the liberty of the designer; they lead to uneconomical designs and impossible spirals for columns 20 in. square and smaller, and especially in places where 2-in. fireproofing is required.

Spiral reinforcement is now being sold at approximately $100 per ton; spiral wire may be obtained with an ultimate strength of about 180,000 lb per sq in. for only about $10 per ton more than common, cold-drawn wire. If one wire tests at 180,000 lb per sq in. and another at 60,000 lb per sq in., the designer certainly should be permitted to use half the weight of the stronger as compared with the weaker. The Committee should explain in detail their reasons for limiting the strength of wires in spirals to 60,000 lb per sq in. Professor Richart has shown [26] that, where high carbon wire spirals were used, the stresses at the time of failure in the wires were 150,000 lb per sq in.

It should be especially noted that the Committee considered the conditions at the ultimate load for the determination of the amount of spiral reinforcement required to balance twice the strength of the protective coat, whereas the permissible stresses in concrete and vertical steel were based on conditions at the working load, after five months timeloading. This is utterly incongruous.

The writer has compared Eqs. 16 and 17 for combined axial compression and bending stress with the following tests and has noted serious discrepancies:

(*a*) Two columns by M. Considère,[8] 16 in. square and 197 in. long, with 4% of vertical steel and with proper companion columns.

(*b*) Sixteen different columns by the Austrian Concrete Committee,[24] 10 in. square, octagonal or hexagonal, with 1.3% to 3.15% of vertical steel and from 118 in. to 177 in. long. Half the columns had 0.7% of spiral reinforcement.

(*c*) Fifty-six columns by Professor M. R. von Thullie[28] with 1.76% of vertical steel and $\frac{e}{t} = \frac{1}{4}$ and $\frac{1}{2}$.

(*d*) Forty-two large columns by Professor C. von Bach and Prof. O. Graf.[29]

(*e*) About forty columns by Professor Richart.[30]

Eq. 17 seems to have no proper basis at all. As has been stated previously, most competent observers have ascribed to the protective shell of a spirally

[28] *Beton und Eisen*, 1916.

[29] Forschungsarbeiten auf dem, Gebiete des Ingenieurswesen, Heft 166–169, 1914.

[30] *Journal*, Am. Concrete Inst., March–April, 1938.

reinforced column a unit strength of not more than 50% of the cylinder strength, whereas this formula leads to permissible stresses in the outside fibers of from 40% to 45% of the cylinder strength.

Elsewhere the writer has given[31] the following simple formula for rectangular and round columns eccentrically loaded, which agrees with all the aforementioned tests:

The permissible load for an eccentricity $e$ equals the permissible axial load divided by $\left(1 + 3.14 \frac{e}{t}\right)$. There is nearly an "exact" agreement with tests for $\frac{e}{t}$ less than about 0.25, for all percentages of reinforcement, and a very good agreement for $\frac{e}{t} = 0.5$ when the vertical reinforcement amounts to from 3 to 4%. For larger eccentricities the *Proceedings* of the American Concrete Institute for 1937 should be consulted.

Paragraphs 865 and 866 propose another unnecessary innovation for the determination of the bending moments in square and rectangular footings. It should be rejected.

*Summary.*—In general it may be stated of this Report that it has unduly emphasized features connected with ordinary building construction. Any engineer seeking guidance for the fine points of design or practice pertaining to reinforced concrete in marine work, conduits, tanks and reservoirs, chimneys, dams, arches, rigid-frame bridges, etc., will be disappointed.

Deflections in many structures become of the utmost importance when factors of safety are as low as recommended by this Report. No advice has been given as to what modulus of elasticity or what "shortcuts" to use when computing the deflections in slabs, T-beams, two-way slabs, flat slabs, ribbed floors, arches, etc. On the other hand, an almost entirely impracticable scheme has been presented in Appendix 2 for the computation of indeterminate moments in continuous beams.

The fundamentals of the science of reinforced concrete seem to be in a state of confusion. The literature has become so vast and contains so many discrepancies and contradictions that the truth is now obscured by a dense fog; and it will take heroic measures to remove it. There does not exist today a complete list of the works or writings of the most eminent pioneers and investigators and builders from which a Committee such as this could find a guide to authoritative information.

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[31] *Transactions*, Am. Soc. C. E., Vol. LXXXVI, 1923, p. 1185; and *Proceedings*, Am. Concrete Inst., 1937, p. 498.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

# PERMISSIBLE COMPOSITION AND CONCENTRATION OF IRRIGATION WATER

## Discussion

BY MESSRS. CARL S. SCOFIELD, WALTER W. WEIR, AND ROBERT S. STOCKTON

CARL S. SCOFIELD,[7] ESQ. (by letter).[7a]—Engineers would like to have a rule to establish the composition of irrigation water. The author recognizes this fact, and the major objective of his paper is to show the difficulty or impossibility of drafting such a rule. The crux of the matter is put into one sentence: "One of the critical points in this entire question is not so much the salinity of water that is applied as the resulting effect that will be produced on the soil solution." In other words, it is the composition and concentration of the soil solution rather than of the irrigation water that determine the productivity of a field or of an area within a field.

It is almost universally true that the soil solution is more concentrated than the irrigation water by which it is replenished. This follows because the soil solution loses water by evaporation and the dissolved constituents do not evaporate, and also because, in general, plants absorb from the soil solution a higher proportion of water than of dissolved constituents. Thus, both evaporation and plant absorption tend to concentrate the soil solution. The input of rain water dilutes the soil solution; and root-zone leaching, either by rain or by copious irrigation, displaces the soil solution within that zone. Thus, the concentration of the soil solution may range up to manyfold that of the irrigation water, and the ratios of the two concentrations may be very different at any time in different parts of the same field or at different times in the same location.

Even in respect to the concentration of the soil solution the criterion as to the upper permissible limit must be a matter of definition. For a given plant in a given climatic environment a certain constituent concentration of the soil

NOTE.—This paper by W. P. Kelley, Esq., was published in April, 1940, *Proceedings*.

[7] Agriculturist, U. S. Bureau of Plant Industry, Washington, D. C.

[7a] Received by the Secretary May 14, 1940.

solution (for example, 500 ppm of chloride) may cause a measurable depression of growth—say a reduction to 80% of normal. For the same plant and environment increasing concentrations cause further depression of growth until at possibly 3,000 ppm the growth may be only 10% of normal. With such a wide range between the concentration that causes measurable injury and one that would kill the plant, there is room for a difference of opinion as to the permissible concentration of the soil solution for a given plant in a given environment. It may be added that among crop plants there are wide differences of tolerance to any salt constituent. To produce equivalent growth depression by the same constituent in two different crop plants may require as much as a five-fold difference in solution concentration. Furthermore, the salt constituents differ greatly in their effect. For example, with some plants boron is at least 100 times as toxic as chloride and there is reason for believing that with some plants chloride is at least twice as toxic as sulfate.

Because of these facts: (*a*) That there is no consistent relationship between the composition and concentration of irrigation water and that of the soil solution; (*b*) that crop plants differ widely in their reactions to the same salt constituent; (*c*) that climatic conditions influence plant reactions to salt constituents; and (*d*) that there is no generally accepted criterion as to what degree of crop injury is "permissible," Professor Kelley has refrained from giving a definitive list of permissible concentrations. In this the writer believes the author has been well advised.

It would be possible to formulate an acceptable list of constituent concentrations for irrigation water below which, for any given soil condition and climatic environment, the danger of salinity injury to crops would be so remote as to be negligible. It would also be possible to make another list of constituent concentrations for irrigation water above which the danger of salinity injury would be almost certain. Between these two lists would lie a broad zone within which "permissibility" would be conditioned by soil type, subsoil drainage, kinds of crops, climatic conditions, and adequacy of water supply.

Engineers would do well to read carefully the fifth from the last paragraph of the paper. In this paragraph Professor Kelley discusses the need of copious irrigation and root-zone leaching where saline irrigation water is used. The thesis thus presented is one that the writer heartily supports.

WALTER W. WEIR,[8] ASSOC. M. AM. SOC. C. E. (by letter).[8a]—The engineer would like to know how much salt may be contained in irrigation water and still leave it suitable for irrigation purposes. Mr. Kelley has very carefully avoided a direct answer to this question, notwithstanding the fact that he is probably as well-informed regarding the chemical analysis of irrigation waters as any one in the West. It is a relatively simple matter to determine the amount of soluble salts present in any given sample of water, and with a knowledge of the environmental and cropping conditions—as well as the physical and chemical composition of the soil upon which the water is to be used—it is

[8] Drainage Engr., Univ. of California, Berkeley, Calif.

[8a] Received by the Secretary July 29, 1940.

possible, with reasonable accuracy, to predict with what safety the water may be used. Generalizations are dangerous, however. For example, the waters used in the Salt River and Imperial valleys are relatively high in soluble salts, particularly chlorin and sulfates, and yet where drainage conditions are good may be used with safety in these localities or on soils such as those of the Imperial and Salt River valleys which are high in calcium. Irrigation waters derived from the Kern, Kings, and San Joaquin rivers, and many of the well waters used in the vicinity of Fresno, Calif., are exceptionally low in total dissolved salts, but, because of the nature of those salts which are present, and the particular soils on which they are used, they have resulted in some injury. In the soils of this area the ratio of calcium to sodium is low, and the continued use of these waters would eventually render these soils unfit for agriculture except for the fact that most of the soils are pervious and drainage is good and water is plentiful. Wherever, on the east side of the San Joaquin Valley, drainage is poor and high water tables have developed, the sodium carbonate in the irrigation water is beginning to show its effect.

Mr. Kelley has shown very clearly that poor drainage, whatever may be its cause, is likely to be a more important factor than the salt content of the irrigation supply. The other important point that he emphasizes is the necessity of using more water for irrigation than is required for plant growth; in other words, leaching beyond the root zone is essential to the continued success of irrigation farming even if the salt content of the water used is low. It must be realized, however, that the use of excessive quantities of irrigation water may leach some desirable salts or soluble nutrients from the soil, as well as undesirable salts, and that the dangers of a high water table and poor drainage are increased as the use of water increases. If the soils to be irrigated are sufficiently pervious to provide good drainage and leaching below the root zone, and sufficient water is used to keep the upper soils leached of excessive accumulations of salts, it is possible to continue to irrigate with waters which contain rather high concentrations of alkali salts. On the other hand, if drainage is poor or if only limited quantities of water are available, so that leaching rarely occurs, even small quantities of salts in the irrigation water may eventually become troublesome. Therefore, it is obviously impossible, from present knowledge of saline or alkali waters, to make a definite statement as to the amount of alkali salt that water may contain and still be safe for irrigation purposes.

ROBERT S. STOCKTON,[9] M. AM. SOC. C. E. (by letter).[9a]—It is desirable that engineers and the courts should take cognizance of the fact that long continued use of water for irrigation on certain lands has a tendency to load the soil with salts deleterious to plant growth.

The accumulation of salts may be fairly rapid as in the case of alkaline unproductive areas that have developed within most irrigation systems within a few years after water is applied, due mostly to rising water tables and the

[9] Cons. Engr., Irrig. and Land Development, Craigantler Ranch, Thompson Falls, Mont.

[9a] Received by the Secretary August 1, 1940.

concentration of salts already in the soil, and only to a slight extent due to salts in the water applied.

As the author states, the quantity of salts in water used for irrigation varies widely; hence the effects, which depend also on the soil, climate, and other factors, may occur in a relatively few years, or generations of users may spread the water before great damage occurs. This slow filling of the soil with salts was the probable cause of the abandonment of many or, perhaps, all of the ancient irrigation systems whose traces may now be found in many parts of the world.

The present knowledge of these facts is of importance in designing new irrigation systems, in laying down the policy of operation and maintenance, and establishing methods of properly using the water available, as well as in protecting values under established systems.

Some lands will no doubt become unproductive in spite of any practical method of preventing or curing the trouble, but, since there is so much more land in need of irrigation than there is water to serve it, there is usually a good chance that the water can be used on new lands with greater precautions and better methods. Since these changes usually occur very slowly, there is a chance to distribute the capital loss over a period of years.

The author emphasizes the use of excess water to carry away excess salts, but this must not be taken as permission to countenance waste of water without the most careful agreement by all authorities that such waste is justified. Unless very carefully supervised, the use of excess water may cause much more alkali trouble by raising water tables and leaching the soils than any other factor. In fact, the great fight of irrigation managers and conservationists is to prevent waste of water with its attending damages and reduction of the area that can be served from any given source.

One may say that the water shall be applied so carefully and skilfully that there shall be, as far as practicable, no upward movement of applied water. This is insured, in large measure, by cover crops, such as grassland meadows, and by prompt and thorough cultivation of all cropped land after each irrigation, so as to establish a mulch and thus reduce evaporation of water from the soil.

The great practical difficulty in the field is in controlling and directing the water users, who are in most cases struggling to make a living and lack time and help to attend to irrigation properly; they feel that they must do things the easiest way and are often wedded to wasteful and unscientific methods. It is most essential, therefore, that court decisions and irrigation engineers and managers shall do all that can be well done to educate the public and enforce those methods that shall best protect land values and production for future generations, as well as for the more or less immediate present.

The probability of the concentration of alkaline salts in soils under irrigation justifies a much greater expenditure for deep drainage than was formerly thought necessary. Many irrigation systems have had to increase the drainage system from time to time, and we may expect, in a large number of cases, that the drainage system will ultimately be nearly as extensive as the water dis-

tribution system. These considerations may help to construct, when feasible, the drainage system before the lands are rendered unproductive and thus save expensive reclamation or loss of land. This necessity for drainage, of course, shows that there is in fact an excess of water used, and the problem is to provide for it and at the same time use water so that there will be as small a waste as practicable, but still providing for a slight downward percolation.

In conclusion it may be said that irrigation engineers should never lose sight of the losses in soil fertility due to irrigation that puts water below the root zone; and further, that if soils are kept rich in nitrogen and humus, very much less water is necessary to be supplied by irrigation, and evil effects are minimized.

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DISCUSSIONS

# TREND IN HYDRAULIC TURBINE PRACTICE
# A SYMPOSIUM

**Discussion**

By K. W. Beattie, Assoc. M. Am. Soc. C. E.

K. W. Beattie,[35] Assoc. M. Am. Soc. C. E. (by letter).[35a]—In this paper Mr. Davis presents an interesting summary of some of the problems encountered in the design of hydraulic turbines. In describing the development of the Kaplan-type turbine he describes the flat efficiency curve which can be obtained when the runner blade angle is adjusted automatically to have a predetermined relation to guide vane opening. To obtain these high efficiencies in actual operation it is essential that this relationship be determined accurately, and this can be accomplished only through the medium of tests conducted on the prototype. Since the correct blade-gate relation differs for various effective heads, such tests should be repeated at different operating heads. These tests on the prototype are easily conducted, using casing piezometers as an index of flow, and should not be neglected if serious losses in operating efficiency caused by improper setting of the runner blades are to be avoided.

Discussing the subject of cavitation testing, Mr. Davis states that tests at the Holtwood laboratory, and others by Professor Spannhake, have indicated that there is no variation in final test results when the test head is varied from 58 ft to less than 33 ft. Tests in the I. P. Morris laboratory demonstrate that cavitation tests may safely be conducted at heads as low as 25 ft, provided that excessive quantities of air are not present in the water. When this laboratory was put in operation for cavitation tests on 11-in. diameter model turbines, the first test to be conducted was on a model that was very carefully machined to be exactly homologous with a 16-in. diameter unit previously tested at the Holtwood laboratory. Fig. 26 shows some of the sigma curves from this test plotted on the same sheet with similar curves from the Holtwood test. In spite of the difference in test heads from 39 ft to 25 ft, and the differ-

Note.—This Symposium was published in November, 1939, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: January, 1940, by Messrs. W. S. Pardoe, and Donald H. Mattern; March, 1940, by Messrs. Lewis F. Moody, and R. E. B. Sharp; April, 1940, by Messrs. Martin A. Mason, and E. Shaw Cole; and May, 1940, by Messrs. Paul L. Heslop, and J. D. Scoville.

[35] Research and Test Engr., Baldwin-Southwark Corporation, I. P. Morris Div., Eddystone, Pa.

[35a] Received by the Secretary July 8, 1940.

ence in the size of units, an excellent agreement is indicated in location of the breaks in the sigma curves.

In regard to the shape of sigma curves, Mr. Davis states that discharge is increased when cavitation occurs, whereas results of tests of some model runners have shown that the discharge may decrease slightly when cavitation

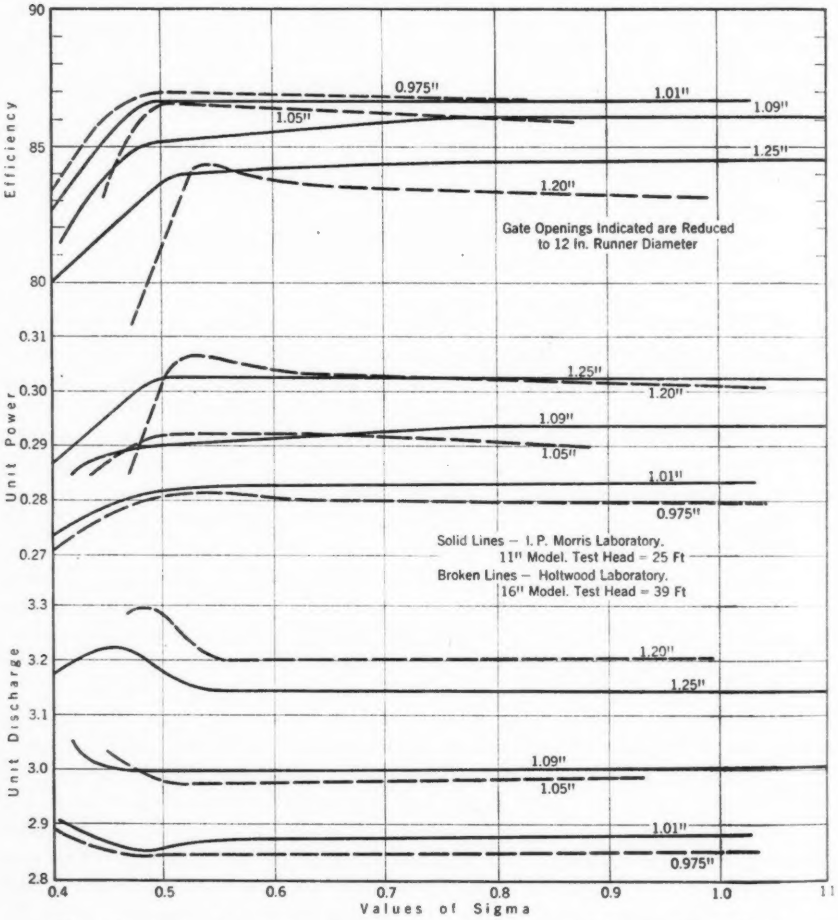


FIG. 26.—COMPARISON OF CAVITATION TESTS OF HOMOLOGOUS RUNNERS MADE UNDER DIFFERENT TEST CONDITIONS

first starts. Tests of a number of different runners made under a wide range of conditions of blade angle and phi have resulted in sigma curves of many different shapes, some of which are as indicated in Fig. 16 of the paper. Figs. 27 and 28 show some other curves, and in these the discharge decreases. In the curves of Fig. 27, the decrease in discharge is not immediately accompanied by a drop in power; so there is a slight rise in efficiency, at low values of sigma.

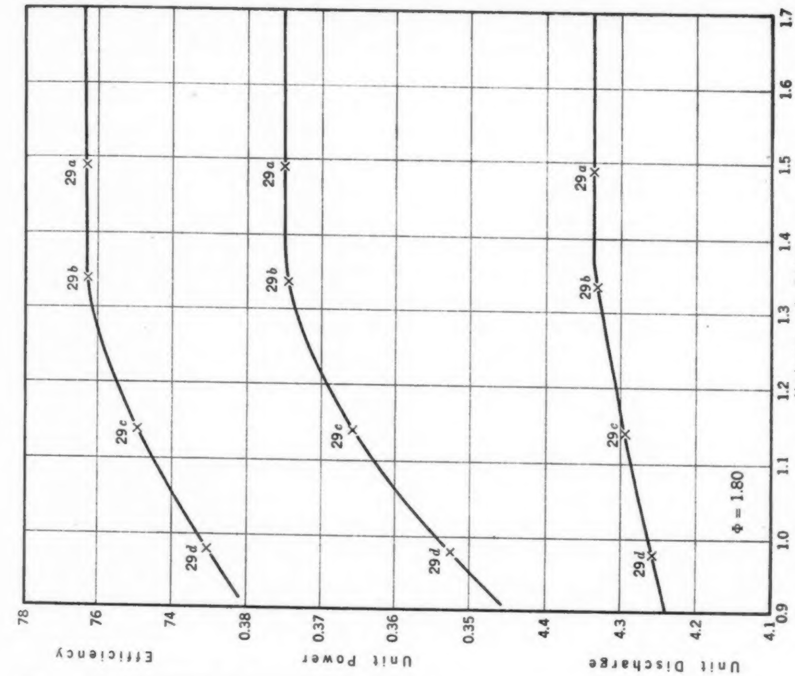


FIG. 28.—SIGMA CURVES WITH POINTS REFERRING TO FIG. 29

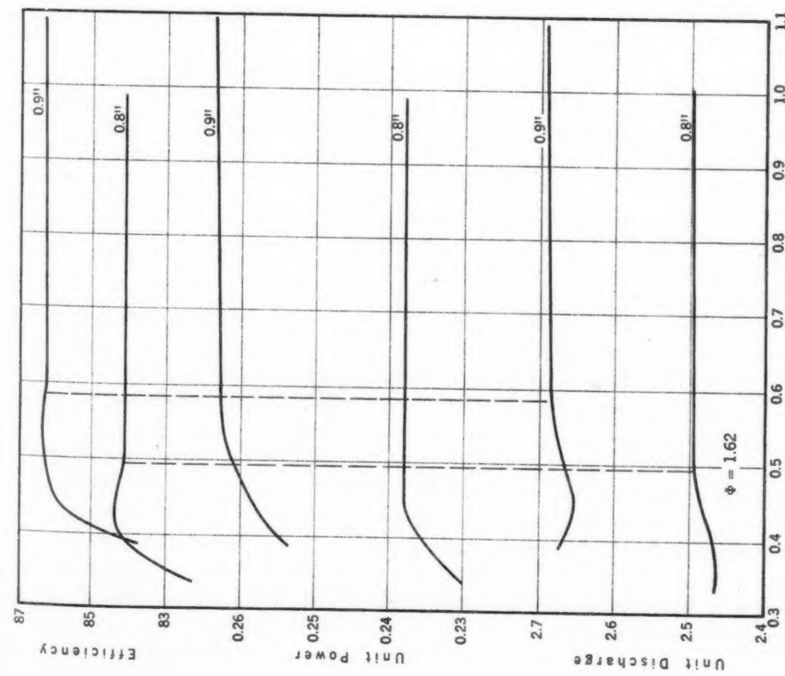


FIG. 27.—CAVITATION TEST—11-IN. MODEL KAPLAN TURBINE

In some cases, this efficiency increase has amounted to several per cent. In a few cases an increase in power has occurred over a narrow range of sigma, near the break.

It will be noted that the curves of Fig. 27 do not show a coincidence of the breaks of the discharge, power, and efficiency curves, and in some cases the spread between these breaks has been greater than that indicated herein. The writer cannot agree that this, of itself, is evidence of faulty design, nor that such a conclusion is the consensus of opinion among hydraulic engineers, as stated by Mr. Davis. It should be expected that cavitation will first occur at some local portion of the blade where the pressure is a minimum and progressively spread as the pressure is reduced, with an accompanying gradual change in turbine performance. To expect general cavitation over the entire area simultaneously is to assume uniformity of pressure over the entire back of the blade, and this is not essential to good design. In the case of Kaplan runners designed to operate efficiently under wide ranges of head and power output, such uniformity of pressure is impossible of achievement.

The criterion which differentiates between good and faulty design is the value of sigma at which the turbine performance is affected, and not the shape of the curves. As stated by Mr. Davis, the critical sigma should be defined as that value at which the first change in performance is noted, whether it is manifested by a change in power or discharge, and even if it is accompanied by an increase in efficiency. The broken vertical lines of Fig. 27 indicate critical sigmas according to this definition.

Very interesting visual observations of the occurrence of cavitation on the backs of model runner blades have been made possible by the provision of windows in the bottom of a model draft tube in conjunction with a powerful stroboscopic light source. The light flashes are synchronized with the turbine shaft, and the source is a vapor lamp producing flashes of very short duration so that the runner is completely "stopped" and photographic time exposures can be made.

Fig. 29 contains photographs of an 11-in. diameter runner operating at 1,600 rpm under a head of 28 to 30 ft, and developing a unit power of 0.375, the value of phi being 1.80. The sigma curves of Fig. 28 were obtained under these same conditions and indicate a break at a sigma of 1.37. In the photograph, light areas on the blades are produced by the presence of vapor and indicate the cavitation areas. In Fig. 29(*a*), the photograph was made at a sigma of 1.49 and small areas of cavitation are discernible at the periphery and close to the hub, although the turbine performance has not been affected. The other photographs are at successively lower sigmas and the progressive spread of the cavitation areas is apparent.

The photographs shown were selected for this discussion because of their clarity rather than for excellence of turbine performance or because they were considered typical. The pattern of cavitation areas is quite different for various runners and different operating conditions. Photographic records of cavitation areas provide information regarding the location of areas on the prototype to be prewelded with stainless steel, and also furnish valuable clues to improvements in design.

Mr. Davis stresses the importance of conducting cavitation tests on completely homologous units. It is agreed that this is an important point as regards the wheel setting and draft tube, but the necessity of including the

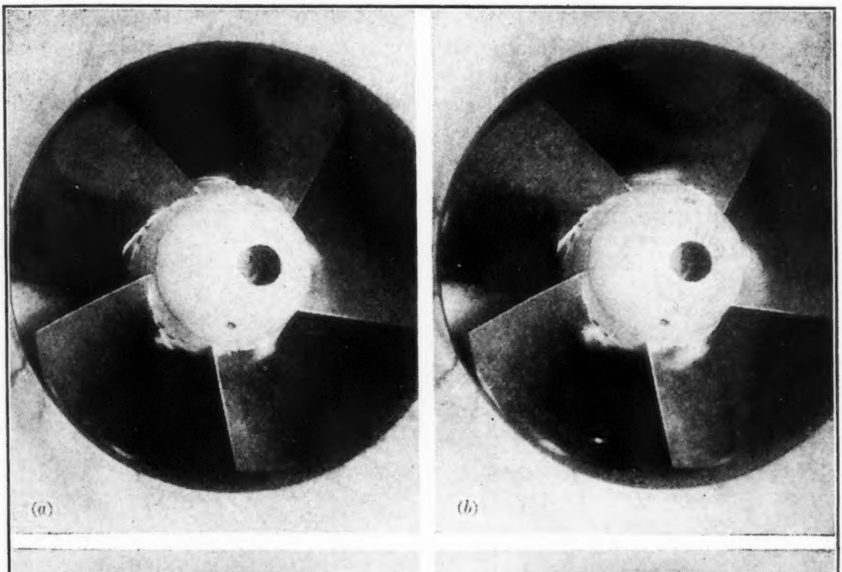


FIG. 29.—RUNNER, 11 IN. IN DIAMETER, OPERATING AT 1,600 RPM, UNDER A HEAD OF 28 TO 30 FT AND DEVELOPING A UNIT POWER OF 0.375 ($\phi$ = 1.80)

intake and casing has not been established. It is doubtful that any disturbance produced in a casing of usual design would have an effect upon cavitation in the runner. One group of tests has been made in the I. P. Morris

laboratory during which the same unit was tested with and without the model casing. There was no difference in the appearance of the sigma curves.

A procedure for relating the results of cavitation tests on the model turbine to performance of the prototype has been employed which differs slightly from that described by Mr. Davis. Critical values of sigma are plotted against unit power, and it is assumed that cavitation may occur at the same unit power on the prototype. Such a curve is shown in Fig. 30, plotted on the same sheet with efficiency. Because of the increased efficiency of the large unit, this method results in a more conservative interpretation than does the method described by Mr. Davis in which unit discharge is the constant.

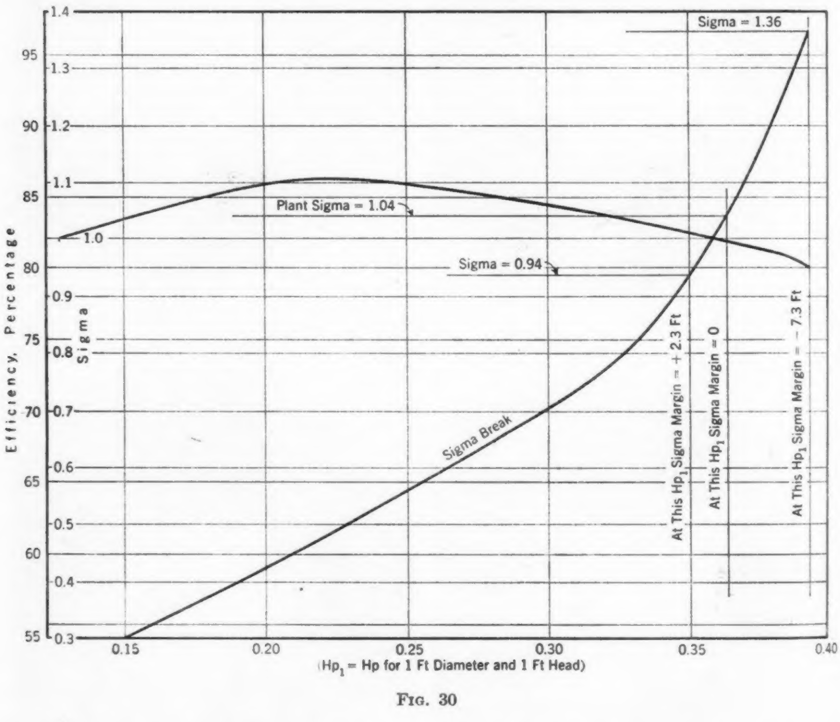


Fig. 30

Referring again to Fig. 30, it will be noted that the curve of critical sigma rises sharply as the maximum power is approached. This demonstrates the importance of limiting the power output of the unit to the value corresponding to the plant sigma. Increasing the output beyond this point involves operation at sigmas well below the critical value and may result in serious damage due to cavitation pitting.

There can be nothing but agreement with Mr. Davis when he suggests that additional study and laboratory work will result in further improvements in performance of hydraulic turbines. The investigation of cavitation characteristics alone offers an extensive field for further research.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# DISCUSSIONS

# MASONRY DAMS

## A SYMPOSIUM

### Discussion

By Messrs. William P. Creager, J. R. Shank, George R. Rich, Robert A. Sutherland, Ross M. Riegel, Paul Baumann, W. A. Perkins, L. J. Mensch, and Lewis H. Tuthill

William P. Creager,[54] M. Am. Soc. C. E. (by letter).[54a]—An excellent résumé of the basic assumptions used in the design of modern masonry dams is presented in the paper by Messrs. Houk and Keener. In engineering, as in most professions, there is seldom exact agreement on many subjects. The writer desires to defend two assumptions of design which differ from those outlined in this paper.

Messrs. Houk and Keener claim that consolidated "silt, sand, or gravel" deposits above concrete dams exert no pressure on the dams. The writer cannot subscribe to that claim. He believes that he needs to present no argument to defend his assertion that coarse "sand and gravel" will exert horizontal pressure no matter how long it has been placed.

As regards the pressure of silt, the authors of this paper, in sole support of their theory, call attention to the Goldbeck cell measurements in the cores of the Miami and Tieton dams to indicate that horizontal pressures did not exceed "hydrostatic pressure" after consolidation had taken place.

Fig. 16 is from data submitted in 1922 by C. H. Paul (6),[54b,55] M. Am. Soc. C. E. A line has been added representing total head of water above any point. It will be noted, as claimed by Messrs. Houk and Keener, that at a depth of 50 ft below water surface, where the material had consolidated, the total pressure of water and soil on the Goldbeck cell was approximately equal to the total head of water above that point.

In order to show that this fact did not prove the lack of soil pressure, it is necessary to investigate the conditions which existed in the core at the time

Note.—This Symposium was published in May, 1940, *Proceedings*.

[54] Cons. Engr., Buffalo, N. Y.

[54a] Received by the Secretary May 20, 1940.

[54b] Numerals in parentheses, thus: (1), refer to corresponding items in the Appendix of the paper.

[55] See reference (6), bibliography, Fig. 4.

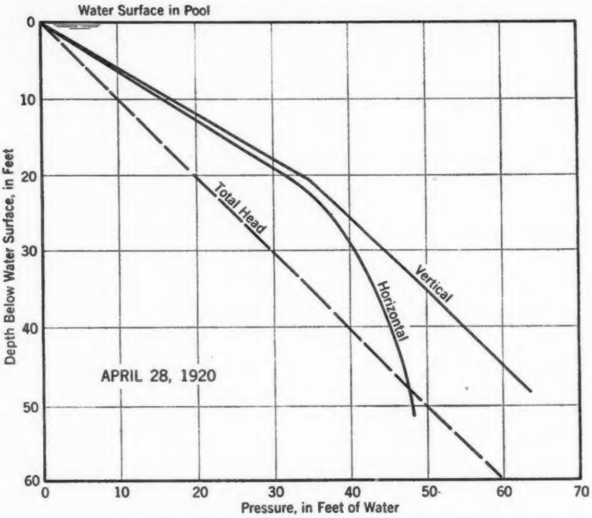


FIG. 16.—PRESSURES IN GERMANTOWN CORE

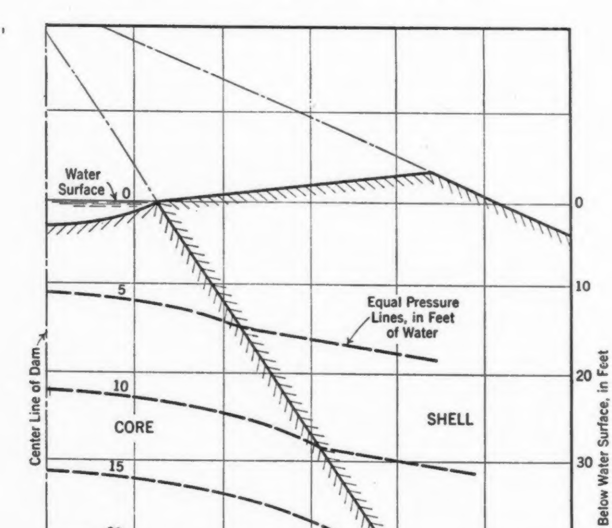


FIG. 17.—WATER PRESSURE IN THE CORE OF AN HYDRAULIC FILL DAM

the measurements were taken. These conditions were probably similar to those indicated in Fig. 17, in which the writer has plotted, somewhat diagrammatically, the lines of equal water pressure (not total pressure) in the core of an hydraulic fill dam. The water is seeping from the pool down through the core and into the shell.

It will be noted that, due to the seepage friction losses, the water pressure at any point is much less than that corresponding to the total head above that point. At a point on the center line, where the depth (and hence the total head) is 50 ft, the water pressure is only 30 ft. That is, the water pressure, alone, was only 60% of the total head. Consequently, since the Miami Goldbeck cells recorded pressures equal to about total head, and since the direct water pressure accounts for only 60% of the total head and therefore accounts for only about 60% of the Goldbeck cell reading, the other 40% must have been soil pressure.

The total horizontal unit pressure of soil and water may be computed at any point as follows: Let $w$ = the unit weight of water; $p$ = the ratio of unit water pressure at any point, as shown by Fig. 17, to the total head of water above that point; $w_s$ = the submerged weight of the soil; $h$ = the depth of water; and $h_s$ = the depth of soil. The total horizontal unit pressure is made up of the following, using Rankine's equation for the ratio of horizontal to vertical soil pressures:

1. The direct pressure of the water,

$$P_w = w\,h\,p \qquad (5a)$$

2. The horizontal component of the submerged weight of the soil,

$$P_s = w_s\,h_s\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \qquad (5b)$$

3. The horizontal component of the weight of water transferred to the soil by friction, or "suspended" water,

$$P_w' = w\,h\,(1 - p)\,\frac{1 - \sin\phi}{1 + \sin\phi} \qquad (5c)$$

Assume for the point in Fig. 17 that $\phi = 30°$; $p = \frac{30}{50} = 0.6$; $w = 62.5$; $w_s = 57$; and $h = h_s = 50$.

| | |
|---|---|
| From Eq. 5a, $P_w = 62.5 \times 50 \times 0.6$ | = 1,875 |
| From Eq. 5b, $P_s = 57 \times 50 \times \frac{1}{3}$ | = 950 |
| From Eq. 5c, $P_w' = 62.5 \times 50 \times (1 - 0.6) \times \frac{1}{3}$ | = 417 |
| Total horizontal unit pressure, in pounds per square foot | = 3,242 |
| The unit pressure (in pounds per square foot) corresponding to total head of water = $62.5 \times 50$ | = 3,125 |
| The ratio of actual pressure to total head of water is $\frac{3,242}{3,125}$ | = 1.037 |

Thus it is seen that, although the direct horizontal pressure of the soil is 950 lb or $\frac{950}{50} = 19$ lb per ft of depth, the total horizontal pressure is barely in excess of hydrostatic as in Fig. 16.

The total water and silt pressure acting on the upstream face of concrete dams depends upon the relative permeability of the silt and that of the foundation. In Fig. 18 the silt deposit and the foundation are assumed to be equally permeable. As the water seeps from headwater to tailwater, the entire head acting on the dam is lost in friction. Fig. 18 shows approximate lines of equal water pressure for this case; 40 ft of the 100 ft of total head is lost through the

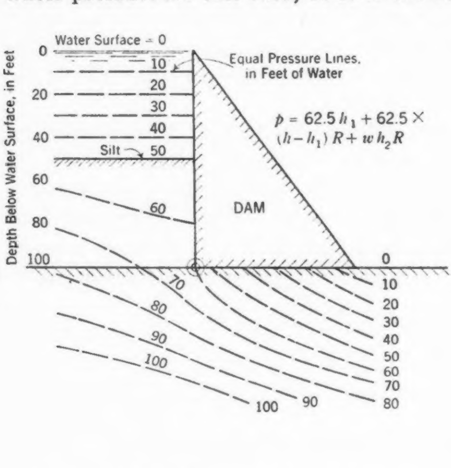


FIG. 18.—SILT HAS THE SAME PERMEABILITY AS THE DAM

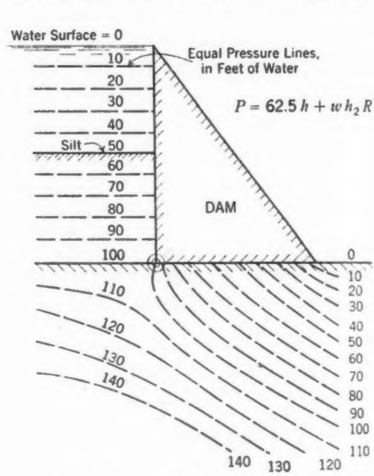


FIG. 19.—SILT HAS RELATIVELY INFINITE PERMEABILITY

silt, leaving a water pressure of 60 ft, or 60% at the base of the dam. Using the units previously adopted, the total horizontal pressure would then be:

| | |
|---|---|
| From Eq. 5*a*, $P_w = 62.5 \times 100 \times 0.6$ | = 3,750 |
| From Eq. 5*b*, $P_s = 57 \times 50 \times \frac{1}{3}$ | = 950 |
| From Eq. 5*c*, $P_w' = 62.5 \times 100\,(1 - 0.6)\,\frac{1}{3}$ | = 833 |
| Total horizontal pressure, in pounds per square foot | = 5,533 |

The unit pressure corresponding to total head of water at the base of the dam is $62.5 \times 100 = 6{,}250$ lb per sq ft. Therefore the total horizontal pressure of 5,533 lb per sq ft is actually less than that corresponding to the total head. The case is analogous to the pressures in cores of hydraulic fill dams previously mentioned.

It also will be noted that, for this case, the uplift pressure on the base of the dam, instead of varying from headwater to tailwater, as is usually assumed, varies from only 60% of headwater to tailwater.

The presence of silt for a case like this would be a distinct advantage. However, the writer believes that the assumed conditions would be very un-

usual. It is problematical just what allowance could be made for cases where it is felt that the permeability of the silt, in comparison with the permeability of the foundation, is low enough to reduce the pressures on the dam.

In most dams on tight rock foundations, the silt must be assumed much more permeable than the foundation. The typical case of a foundation so impermeable that the friction loss in the silt should be neglected is given in Fig. 19. The conditions are assumed to be exactly the same as in Fig. 18 except that the silt deposit is assumed to be infinitely permeable in comparison with the foundation. Therefore, no head is lost due to seepage through it, and the water pressure at any point on the dam corresponds directly to feet of head on that point.

Using the units previously adopted, the total horizontal pressure (in pounds per square foot) would be:

$$\begin{array}{lll}
\text{From Eq. } 5a,\ P_w & = 62.5 \times 100 \times 1.0 & = 6{,}250 \\
\text{From Eq. } 5b,\ P_s & = 57 \times 50 \times \frac{1}{3} & = \phantom{0}950 \\
\text{From Eq. } 5c,\ P_w' & = 62.5 \times 100\ (1 - 1)\ \frac{1}{3} & = \phantom{000}0 \\
\hline
\text{Total horizontal pressure} & & = 7{,}200
\end{array}$$

The water pressure is not diminished by the silt, and the silt pressure is appreciable, amounting to 19 lb per ft of silt depth. The writer believes that the permeability of silt may be negligible in comparison with that of most rock foundations, that therefore the pressure of silt is not negligible, and that it must be considered in the design of concrete dams.

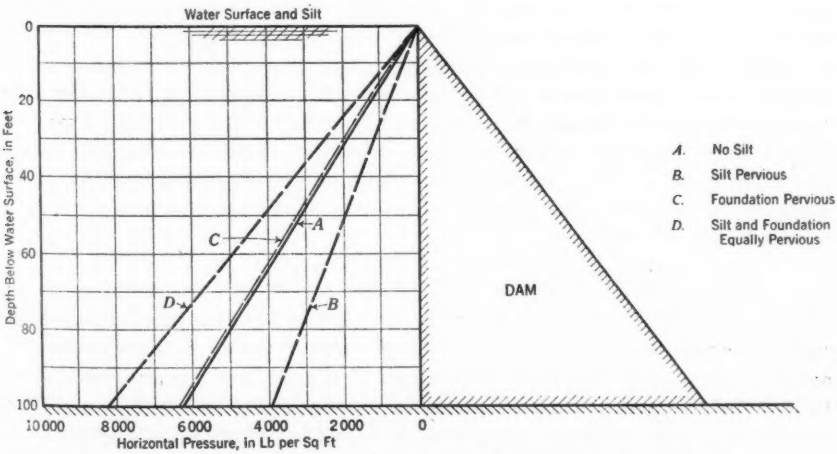


FIG. 20.—UNIT HORIZONTAL PRESSURES IN POUNDS PER SQUARE FOOT

Fig. 20 indicates the horizontal unit pressures of silt and water under certain reasonable assumed conditions, for the pond completely filled with silt. The differences in pressures are quite marked:

Line $A$: No silt.

Line $B$: Foundation very pervious compared with the perviousness of the silt—a very unusual case.

Line $C$: The foundation and silt are equally pervious—also an unusual case.

Line $D$: Silt very pervious compared with the perviousness of the foundation —the usual case.

The method for determining the strength of straight gravity dams against sliding, which Messrs. Houk and Keener state has been used "until a few years ago," may be written

$$\text{Factor of safety} = \frac{S_1}{P_h} + \frac{k\,P_v}{P_h} \qquad (6)$$

in which $\frac{P_v}{P_h}$ = the ratio of the vertical to horizontal forces—that is, the reciprocal of the "sliding factor" used by Messrs. Houk and Keener; $k$ = the friction coefficient; and $S_1$ = the usually unknown, and hence neglected, shearing strength developed by keying and bond.

It was specified that, for $S_1$ equal to zero, the factor of safety must exceed unity.

Messrs. Houk and Keener state that, since 1933, the Bureau of Reclamation has been using a different, or Henny, method; but that method also employs Eq. 6. The sole difference is in the fact that, according to the Bureau method, the factor of safety must be greater than 5 whereas, according to the original method, it must exceed unity when $S_1$ equals zero. Both of these methods are useful, the Bureau method being adapted to cases where the dam could not possibly slide without shearing and the original method being adapted to cases where the shearing strength is unknown.

In the writer's opinion, the shearing strength of many classes of horizontally bedded rock foundations is unknown since it is impossible to determine the extent of weaknesses in such bedding planes. In such cases the original method should be used and the writer advocates using for $k$ the friction angle of well-dressed specimens. In addition, if the shearing along a weak bedding plane would result in quite a flat surface, a "toe-hold" or some other form of anchorage is necessary.

J. R. SHANK,[56] M. AM. SOC. C. E (by letter).[56a]—In the Symposium on "Masonry Dams," the paper by Mr. Tyler is an excellent chapter for a possible comprehensive text on the construction of masonry dams or heavy masonry construction in general. It is comprehensive, concise, and generally well written. No one, save he who has been intimately connected with some large project, can take exception to anything said, and his exception is very likely to arise from some specialty of that particular project. Any one reading this paper will have his own information nicely summarized or averaged, or he will have increased his knowledge considerably.

[56] Prof., Civ. Eng., and Asst. Director, Eng. Experiment Station, Ohio State Univ., Columbus, Ohio.

[56a] Received by the Secretary June 14, 1940.

George R. Rich,[57] M. Am. Soc. C. E. (by letter).[57a]—Against the background of their wide experience with the Bureau of Reclamation, Messrs. Houk and Keener have prepared a comprehensive survey of the salient features of modern dam design; and their concluding statement justly emphasizes the complete dependence of all stress analyses upon the one fundamental assumption that the dam is a homogeneous, uniformly elastic structure, free from appreciable shrinkage cracks. Because the presence of these random shrinkage cracks serves to vitiate much of the refinement in computations based upon the elastic theory, and in serious instances to invalidate even the simplest stress determination, reduction and elimination of such defects appear to present the most important current challenge to engineers engaged in dam construction.

Predicated directly upon the primary assumption of an intact homogeneous structure, abundant analytical technique is now available for computing, to a nice degree of refinement, the stresses resulting from the weight of structure, the hydraulic forces, and other readily determined loadings. In fact, development in these particular phases of dam engineering appears to the writer to have far outdistanced much-needed complementary research in the analysis of stresses resulting from setting temperature and shrinkage effects.

In dealing with the topic of temperature and shrinkage cracks Messrs. Houk and Keener have outlined the important pioneer work of the Bureau of Reclamation in the adoption of low-heat cement, artificially cooling the concrete, constructing massive structures in smaller blocks, and grouting the component units to form a monolithic structure.

The extent to which artificial cooling in dams is economically justified naturally depends upon the size of the project and the progress rate required by the construction schedule. In the case of Boulder Dam it was necessary to pressure-grout the vertical joints between sections to insure development of the requisite horizontal arch action. To be effective, such grouting must obviously be deferred until the major portion of the chemical heat of setting has been dissipated; and to accomplish this dissipation in a concrete mass of such unprecedented size in a reasonable time, it was essential and economical to circulate refrigerated water through an extensive system of cooling pipes distributed throughout the structure.

In constructing the Hiwassee Dam of the Tennessee Valley Authority (TVA) the crack-prevention measures adopted were, in the order of their estimated relative importance: (1) The use of low-heat cement, (2) a slow pouring program, (3) a low proportion of cement in the concrete mix, (4) artificial cooling of concrete after deposition in the forms, and (5) the use of artificially cooled mixing water during the summer.

A basic pouring rate of one 5-ft lift every fifth day was adopted for the dam as a whole, with the stipulation that for a distance of 10 ft above level foundations the rate would be one 2.5-ft lift every three days. Locations where the foundation rock slopes steeply or where large local cavities occur in the foundation are focal points for the development of highly objectionable

[57] Head Mech. Engr., TVA, Knoxville, Tenn.

[57a] Received by the Secretary August 5, 1940.

vertical cracks in planes parallel to the axis of the dam. In such critical locations setting temperatures were controlled by one of two methods—either by a combination of shallow lifts and long exposure (such as a rate of one 2.5-ft lift every eight days) or by circulating river water at natural temperature through a system of pipes embedded in the concrete. Satisfactory temperature control was readily attained by either method and the choice was largely one of economy for each particular case. In rare instances where other than standard lifts were unavoidable, artificial cooling by means of embedded pipes was specified.

The proportion of cement in the concrete mix was initially 0.85 bbl per cu yd, but subsequently it was found feasible to reduce the cement content to 0.80 bbl per cu yd. In summer it was advantageous to cool the mixing water artificially from a natural temperature of about 85° F to 40° F.

The most striking benefit from the use of artificial cooling occurred in a case where it was found necessary to defer pouring certain blocks of the dam to permit passage of the natural flow of the stream during construction, and to carry pouring of the adjacent blocks to a height of about 100 ft. Thus, during setting of the concrete, the block immediately adjacent to the opening was exposed to the atmosphere on one face, while the inshore face was in contact with neighboring recently poured blocks and maintained at an elevated temperature owing to the setting heat of the concrete. On the basis of previous practical field experience, as well as investigations by the theory of elasticity, it was desired to maintain a straight-line temperature gradient through the block to prevent the formation of shrinkage cracks. This was accomplished, readily and inexpensively, by setting resistance thermometers at intervals in the body of the block, pumping natural river water through a system of cooling pipes differentially spaced, and regulating the flow of water through the various sections by means of manually operated valves, so as to approach the desired straight-line temperature gradient. The results were generally very satisfactory, and cracking in the exposed face kept within satisfactory limits.

Since the elimination of shrinkage cracks is becoming recognized as the outstanding difficulty of dam construction, it appears essential as a general principle to provide facilities for eliminating shrinkage defects to the full extent found economically justifiable for the particular project.

With respect to loading conditions, the writer shares the opinion of Messrs. Houk and Keener that advances in earthquake analysis constitute the major current development. However, as a result of recent studies involving impounding structures of widely varying types for the TVA, he is convinced that more consistent results and greater economy, without sacrificing safety, will be obtained by predicating the analysis upon a properly selected period and amplitude for the ground motion and calculating the extent to which the response of the structure approaches resonance, rather than by applying a more or less arbitrary lateral loading of 0.1 gravity.

Instrumental observations by the U. S. Coast and Geodetic Survey indicate that the maximum amplitude to be expected in rock foundations in the United States is about 0.25 in., and that the principal damage results from foundation

movements having amplitudes of this magnitude in combination with periods in the order of about 1 sec. Substitution of these values in the basic simple harmonic-motion equation will give an acceleration of about 0.03 gravity, which agrees with the instrumental records.

Recorded movements in rock foundations having high accelerations in the order of 0.3 gravity in combination with periods of 0.1 sec were found to have caused no appreciable damage. One of the reasons for this will be apparent from again substituting these values of acceleration and period in the simple harmonic-motion equation. The corresponding foundation amplitude will be found to be only 0.03 in. Since the natural period of vibration of many typical massive gravity dams will be found to be in the order of 0.10 sec, the question naturally arises why resonance effects do not prove more disastrous in the high acceleration, low frequency brackets.

The explanation follows readily from a study of the standard equation for forced vibrations with viscous damping—that is, with damping proportional to the first power of the velocity and caused by the internal friction of the concrete of the dam. It will be found that damping has a marked influence upon amplitudes in cases where the resonance ratio approaches unity, and only an incidental effect for other resonance ratios. In addition, it will be noted that, for even the condition of theoretically perfect resonance, the magnification factor, which measures the ratio of dynamic to static deflection, will seldom exceed a small finite value in the order of 2.0 for the short natural periods and damping coefficients applicable to ordinary gravity dams.

There is a second factor that will explain why actual magnifications in the high acceleration, short period range do not even remotely approach theoretical damped resonating values—namely, the fact that earthquake vibrations are transitory and that the particular precise forced period required for sharp tuning or resonance is repeated for only a few cycles. Possibly many cycles would be required to build up the amplitude of the structure to the theoretical damped maximum.

Having accepted a period of 1 sec and an amplitude in foundation rock of 0.25 in. as the most rational general basis for earthquake-resistant design, it is interesting to investigate the typical range of standard dams by dynamic methods. Since the natural period of such dams is in the order of 0.10 sec, the resonance ratio, and consequently the magnification factor, will be small and the acceleration of all parts of the dam will be essentially the same as the foundation acceleration—namely, 0.03 gravity. It appears, therefore, that in the United States gravity dams of average proportions would have ample margin of safety against earthquake disturbance if designed for an equivalent inertia loading of 0.05 instead of 0.10 gravity, and since this same acceleration enters directly into the Westergaard formula (15) commonly used for determining reservoir superpressures resulting from earthquake shocks, a proportionate reduction should be affected in the lateral hydraulic loading.

It is in dealing with lighter, more flexible types of hydraulic structures, such as arch dams, power-station substructures, buttress dams, and high spillway piers, that the dynamic method of calculation is most superior to the

arbitrary inertia load method. The greater the flexibility of the structure, the more pronounced will be the error of assuming that all parts of the structure have an acceleration equal at all instants to the acceleration of the rock foundation. For flexible structures the use of the dynamic method is essential to obtain deflection curves that do not conflict with common observation and experience.

For reinforced-concrete structures the advantages of the dynamic method of analysis are most marked. Before drawing conclusions in any particular case, it is advisable to check the first and second harmonics as well as the fundamental mode of free vibration. However, as a general rule it will be found that an appreciable saving in reinforcement will be realized; that the reinforcement provided will be distributed more uniformly throughout the length of the structural members; and, most important of all, that the steel will be rationally distributed to sustain earthquake shocks.

It is suggested that dams and related structures on rock foundations in the United States be designed for a foundation amplitude of 0.25 in. and a period of 1 sec, and that such structures be proportioned by the dynamic method based upon the ratio between the assumed forced period of vibration of the foundation and the natural period of the dam.

Mr. Lieurance has prepared a very informative paper that may well serve as a working manual for the design of arch dams. The tables represent an enormous amount of labor, and in most cases will be entirely adequate for preparing the final design.

With respect to articulations or flexible joints in arch dams, the writer is pleased to endorse Mr. Lieurance's opinion that it is undesirable to sacrifice any part of the structural strength of the dam in order to simplify its analysis. Following general practice for similar heavy concrete structures, it is the firm belief of the writer that the most dependable arch dam, conforming most closely in the finished and loaded condition to the design computations, will be obtained by avoiding all articulations and by pressure-grouting the radial joints to full and complete bearing after the major portion of the chemical heat of setting has been dissipated. In the case of large structures, such as Boulder Dam, it will be found economical to dissipate this heat by artificial cooling to meet the requirements of the construction progress schedule. To compensate for residual undissipated heat remaining in the concrete, or for any necessary similar adjustment, the arch dam may be deflected upstream any practicable predetermined amount by selecting the proper grouting pressure, so that the completed dam under its combined loading will be a monolith conforming to the elastic computations and having optimum stress distribution.

Mr. Lieurance has given a clear exposition of the various critical loading conditions that measure the adequacy of design at the abutments upon which the integrity of the arch elements is dependent. The effect of building the upper portion of the dam to greater than normal width to accommodate a roadway may be inferred from a study of any typical hydrostatic load division diagram. In representative cases the upper arch rings carry not only the entire local hydrostatic load, but, because of their superior stiffness, also serve as partial top supports for the vertical cantilever elements, so that the diagram

of load tributary to the cantilever elements shows a change in algebraic sign at a point about two thirds the height of the dam. Thickening the upper arch rings aggravates this effect and, in addition, through the medium of tangential shearing stresses, tends to restrain deflection of the adjacent lower arch elements, thus further increasing transfer of load horizontally to the abutments at the higher levels.

With respect to the selection of the economic central angle, Mr. Lieurance cautions against empirical rules, and stresses the desirability of establishing this feature to conform with abutment and foundation conditions of the particular project. The suggestion that the first preliminary examination of stresses be made on an arch section at about two thirds the height of structure again follows readily from an inspection of the hydrostatic load distribution diagram. It will be recalled that in most instances, because of the excess stiffness of the top arches as compared with the top of the vertical cantilevers, the load on the vertical cantilever elements changes from downstream to upstream in direction at about two thirds the height of dam, so that at this point the entire hydrostatic load is carried by the arch elements.

The tables have been so well prepared that there is little opportunity to do more than remark that their use is restricted to symmetrical circular arches and foundation conditions free from singularities.

In investigating thick arches of relatively sharp curvature, it may be desirable to take account of the fact that the neutral axis of the arch does not coincide with the gravity axis. This may be accomplished readily by the well-known Cain-Jakobsen formulas (32).

The tables presented by Mr. Lieurance may be used directly in the determination of earthquake stresses by means of conventional equivalent inertia loadings. Although much research will be necessary to develop the requisite technique, it is suggested that future pioneering be directed toward adapting the dynamic theory of earthquake-resistant design to arch dams. In view of the refinements attempted in connection with foundation deformations, torsion, tangential shear, and Poisson's ratio effects, it appears inconsistent to predicate earthquake stresses, which in many instances may be of comparable magnitude, upon a theory that is merely a convenient fiction when applied to flexible structures in which the magnification factor exceeds unity. The dynamic theory of vibration affords rational solutions for both the longitudinal and transverse forced vibration of fixed-ended beams. It is anticipated that eventually such solutions may be adapted to cover the case of earthquake shocks in directions either parallel, or at right angles, to the axis of the arch dam. Should the mathematical analysis prove cumbersome, valuable assistance might be obtained from shaking-table tests. For actual design the final results should be expressed in simple, convenient form, possibly in the form of uniform loads that would vary for different structural proportions; but the basis for the loading should be rational. It should be postulated upon the critical destructive range of periods and amplitudes of the ground motion at the site, and should involve the magnification factor of the structure or, in other words, the measure of the degree in which the particular structure may approach resonance. This method is attaining recognition as the only satis-

factory approach to earthquake-resistant building design and should logically be extended to include hydraulic structures.

The familiar Westergaard formula (15) for hydraulic superpressures during earthquake shocks should be reconsidered concurrently and possibly modified in the case of the thinner arch dams. As is well known, the original theory is based upon the assumption that the magnification factor for the dam is unity, or, in other words, that all parts of the arch dam have the same acceleration as the foundation rock. This assumption is probably sufficiently correct in the case of typical gravity sections, but for flexible arches some corrections may be necessary. Incidentally, any engineer who uses the Westergaard formula should not fail to read the discussion of the Westergaard paper by Boris A. Bakhmeteff, M. Am. Soc. C. E. Professor Bakhmeteff develops the equation against the familiar background of the theory of water hammer in pipe lines, and his clear presentation of the analogy in the case of negative wave reflections has frequently helped the writer to avoid misapplication of the Westergaard expression.

The basic mathematical research for the foregoing investigations will quite possibly be very complicated and laborious; but the final result, to be of any use in practical designing, must be simple and readily usable. The design tables given by Mr. Lieurance are an admirable example of the reduction of extremely involved analysis to simple workable form. It is hoped that ultimately earthquake analysis for arch dams may be rationalized and presented in similar tabular form.

In conclusion it should be remembered that in spite of the powerful methods available there still remain numerous possible combinations of effects produced by thermal stresses, plastic flow, and water soakage that must be covered, for the present, by a liberal margin of safety, and, finally, as stated by Mr. Lieurance, that designs to meet unusual conditions should be undertaken only by engineers having wide experience with arch dams.

ROBERT A. SUTHERLAND,[58] ASSOC. M. AM. SOC. C. E. (by letter).[58a]—An extremely valuable addition to the literature on the subject of arch dams is afforded by Mr. Lieurance's paper, and the thanks of the profession are due to the author, especially for the excellent tables which accompany it. These tables enable considerable work in the design of an arch dam to be accomplished with a minimum of fatigue and in a minimum of time.

The method of interpolation suggested by Mr. Lieurance for finding values intermediate between tabular values seems to be substantially correct for most cases arising in practice. However, a warning should perhaps be added that in certain cases interpolation may give incorrect values, as may be seen by inspection of Fig. 21, which shows a plot of a few values. This difficulty may be readily overcome by interpolating by means of curves covering the range of values which depart appreciably from a linear variation.

The shortening of labor in computation to which Mr. Lieurance has so notably contributed may be extended somewhat further. A simple alinement

[58] Designing Engr., Harza Eng. Co., Chicago, Ill.

[58a] Received by the Secretary August 9, 1940.

diagram such as Fig. 22 may be constructed to suit the requirements of the individual user and to facilitate the computation of two quantities—first, the sum of the stresses at extrados and intrados, and secondly, the difference of these stresses. The sum of these stresses is proportional to the thrust, and the difference is proportional to the moment, on the usual assumption of linear stress variation. Hence, by adding the two quantities so found, and dividing

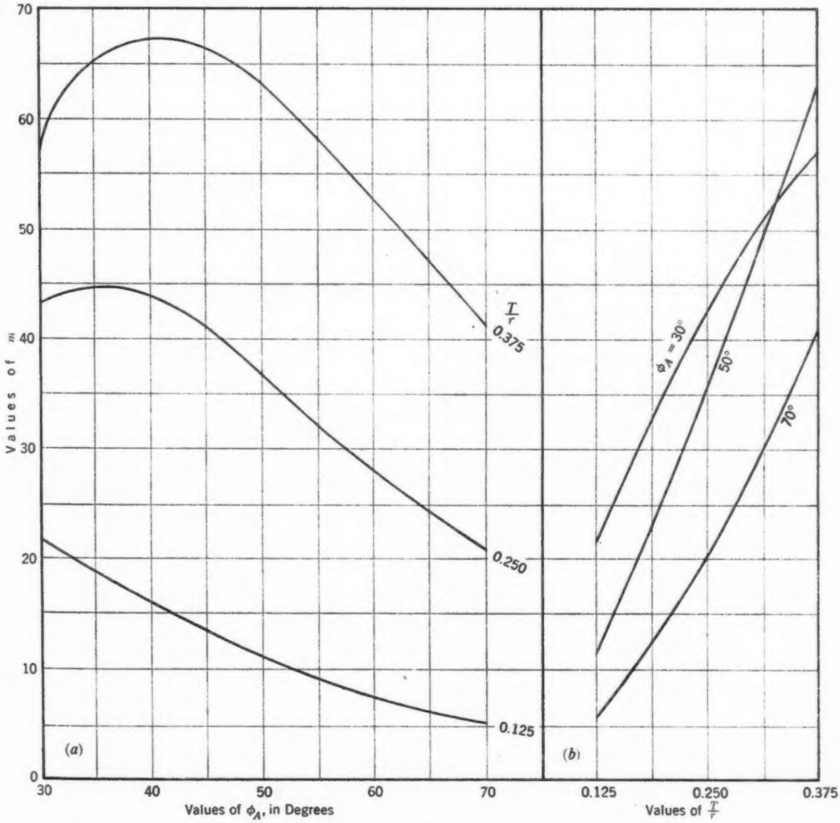


Fig. 21

by two, one stress may be found; and by subtracting the second quantity from the first and dividing by two the other stress may be found.

Alternatively, Tables 3 to 9 could be extended to give stresses directly for the various loadings, angles, and ratio of thickness to radius as tabulated by Mr. Lieurance. The deflection tables may also be extended to express deflections as a percentage of radius, if a certain modulus of elasticity is assumed, which may reasonably be done. For many purposes, such tables are valuable in giving stresses directly, with the absolute minimum of labor.

Several years ago the writer determined values of stresses resulting from the Guidi formulas,[59] and curves representing a few values are submitted as Fig. 23. Stresses determined from these curves have been checked repeatedly against stresses computed from the original Cain formulas and have been found

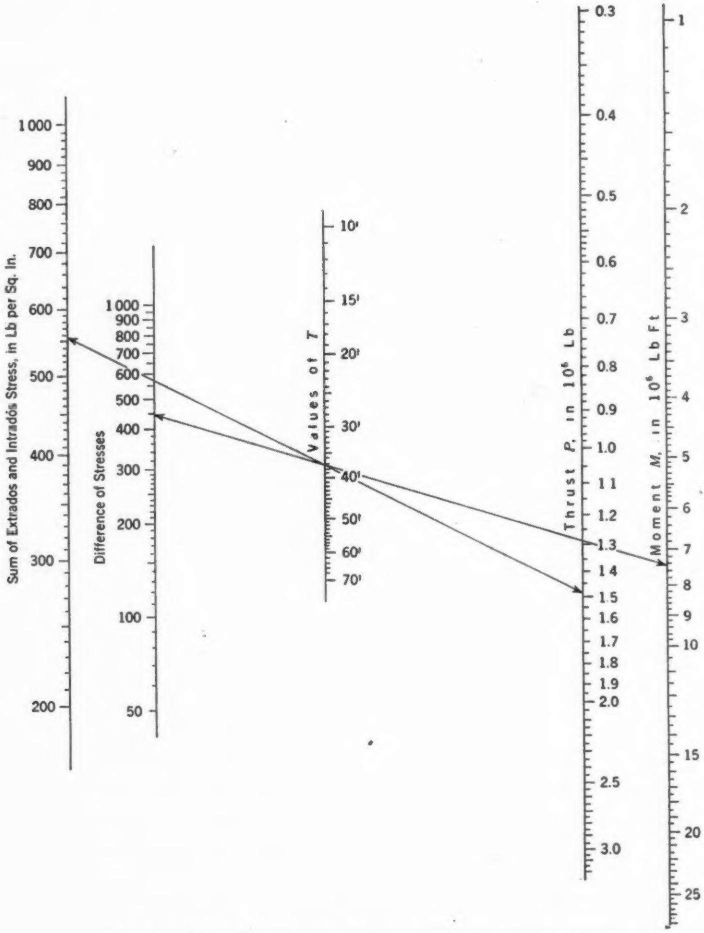


FIG. 22.—ARCH STRESSES IN TERMS OF THRUST AND MOMENT

to check exactly in most cases. Extensive curves based on the revised Cain formulas have been published by Frederick H. Fowler,[60] M. Am. Soc. C. E. Similar curves could be computed readily from Mr. Lieurance's tables, in which abutment deformation is taken account of (which is not the case in the Guidi or Cain formulas), and deflection curves giving deflection in terms of radius

[59] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1623.
[60] *Loc. cit.*, Vol. 92 (1928), p. 1512.

could be computed also. Using such curves, it would be possible by a suitable combination of loadings to translate various forms of effective arch load directly into arch stresses and deflections. The writer appreciates particularly

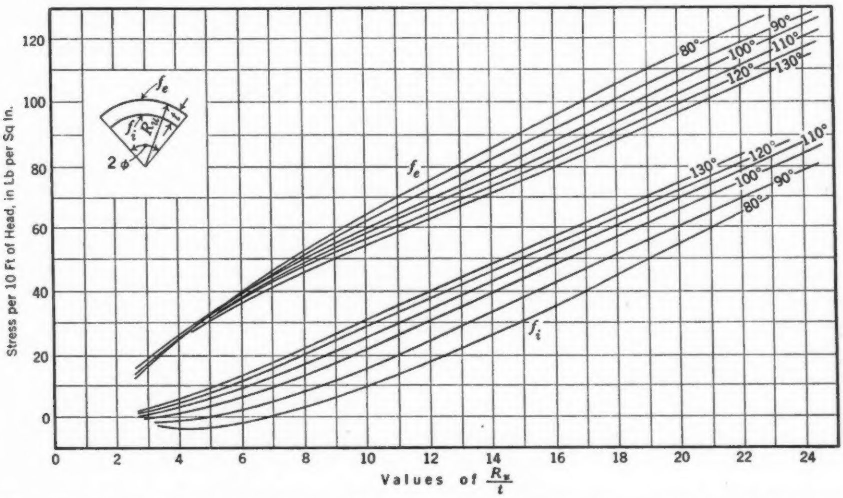


FIG. 23.—CROWN STRESSES IN CIRCULAR ARCH WITH UNIFORM RADIAL LOAD FROM GUIDI FORMULAS

the deflection tables, as they readily enable the deflections for many forms of non-uniform loading to be obtained with a fraction of the effort required if one makes use of influence lines.

TABLE 13.—COMPARISON OF CROWN EXTRADOSAL STRESSES AT A FEW ELEVATIONS IN DIABLO DAM BY DIFFERENT METHODS OF COMPUTATION

| Method | Elevation, in Feet | | | | | Notes |
|---|---|---|---|---|---|---|
| | 1,140 | 1,100 | 1,060 | 1,000 | 960 | |
| Jorgensen..... | 494 | 525 | 577 | 586 | 481 | Mathematical computation from original Cain formulas, using tables of predetermined values of trigonometric functions noted on Jorgensen's Fig. 4 as $C_1$, $C_2$, and $C_3$. |
| Sutherland.... | 491 | 525 | 584 | 582 | 480 | From stress curves interpolating visually for angles only; these curves are based on Guidi formulas. |
| Fowler........ | 481 | 494 | 541 | 533 | 387 | From stress curves, interpolating visually for thickness ratio only; these curves are based on revised Cain formulas. |
| Lieurance..... | 503 | 542 | 623 | 618 | 538 | Using tables of $h$ and $m$ and interpolating arithmetically for both angles and thickness ratios. Stresses are from the alinement diagram from $H$ and $M$. The Lieurance tables include the effect of abutment yield on stated assumptions. |

A comparison will now be made between stresses obtained, first, by direct computation from the original Cain formulas, second, from Fig. 23, third, from Mr. Fowler's curves based on the revised Cain formulas, and last from Mr. Lieurance's tables. In the latter case use will be made of Fig. 22 as a time-

saving device. To the writer's knowledge, the direct computation from the Cain formulas, even when using previously prepared tables of certain trigonometrical functions, is a matter of two or three hours' work for each elevation. The other methods require minutes only. For the sake of example, use is made of a tabulation[61] of arch stresses in the Diablo Dam, to which reference can be made, if desired. Table 13 shows a comparison of the four methods, confined for economy of space and time to only a few elevations and to crown extradosal stresses only. It will be noted that the original Cain formulas and the Guidi formulas give almost identical results, the revised Cain formulas give considerably lower stresses, and the effect of abutment deformation gives higher stresses. Comparing only the Fowler and Lieurance results, as being based on the most up-to-date data of their respective classes (without and with abutment deformation), it is seen that in the example cited the crown extradosal stress is increased by abutment deformation by from 22 to 151 lb per sq in. This emphasizes the absolute necessity of taking account of abutment deformation, and the tables provided by Mr. Lieurance give a very valuable means of doing this.

Ross M. Riegel,[62] M. Am. Soc. C. E. (by letter).[62a]—Some aspects of the paper by Messrs. Paul and Jacobs are, perhaps, entitled to extra emphasis. The writer feels that one aim of the process of foundation treatment should be to develop shearing strength, as well as bearing strength, at and below the contact. He sees the quality of resistance to lateral movement as a function of shearing strength in the foundation and in the concrete placed upon it. For this reason he regards the shear-friction factor used by the Bureau of Reclamation and others as representing most truly the capacity to resist movement when proper constants are used in determining the factor.

The foundation treatment should be designed to secure shearing strength at the contact. Equally the same quality must be secured in the foundation itself. The process of cleaning out and grouting horizontal seams, referred to by Messrs. Paul and Jacobs, increases the shearing strength of the foundations, as well as the bearing strength, and finds justification therein. Other devices, such as anchor walls and deep seating of the structure, are aimed at the utilization of shearing strength to develop resistance against lateral movement.

Messrs. Paul and Jacobs refer to the interpretation of borings and the necessity of proper interpretation, particularly in the matter of the parts of the record usually labeled "no core." The most competent and continuous supervision possible should be a feature of dam site exploration, so that the significance of core losses can be determined with the greatest certainty. The engineering geologist on the job is worth two or three such men looking over cores when the record is "cold"; and it should be the function of this man to ascertain the reason and significance of loss of core as and when it occurs.

Frequently, the loss of core may be avoided, however, by improved processes. The writer recalls a large number of 2-in. diamond-drill cores from a

[61] *Engineering*, London, July 7, 1933, pp. 2–6.

[62] Head Civ. Engr., Design Dept., TVA, Knoxville, Tenn.

[62a] Received by the Secretary August 14, 1940.

shale dam site in West Virginia in which the loss of core from apparently sound shale was as great as 20%. Later, a 5-in. diamond drill was installed, and full recovery secured. Incidentally, these cores were preserved under water after removal from the hole, and thus disintegration in the air, a common phenomenon with such rock, was prevented. The writer deems it justifiable to strive constantly for ever larger sizes of core, not because the recovered rock is more significant, but because the gaps require less explanation or speculation.

The tendency to larger holes has brought about the 36-in. and 40-in. drills described by Messrs. Paul and Jacobs. For the study of the actual structure of a foundation, these are most valuable tools. The direct evidence afforded by an inspection of the interior of such a hole is convincing and authoritative. The writer recalls a very expensive shaft excavated into rock for exploratory purposes in which the desired results could have been achieved by the use of one of these machines at a fraction of the cost; but the machine was not available. The Tennessee Valley Authority used one of these drills at Norris Dam,[63] and the results were so useful that similar machines are now used on all projects.

In conclusion, Messrs. Paul and Jacobs are to be complimented on the manner in which they have covered the subject. Their paper is a most valuable résumé of the essentials of foundation treatment for dams.

PAUL BAUMANN,[64] M. AM. SOC. C. E. (by letter).[64a]—Mr. Steele's paper on "Construction Joints" is perhaps the most complete one so far presented on this subject. It is based generally on the opinions of engineering authorities from the four corners of the earth and particularly on the wealth of experience acquired by the U. S. Reclamation Service in the course of the design, construction, operation, and maintenance of a great number of major dams of various types. Among these Boulder Dam, with its majestic height of 726 ft above its foundation, is unlikely to be infringed upon for a long time to come. In addition to the wide experience upon which this paper is based, its value is enhanced by its broad and open-minded presentation, all of which render its discussion difficult indeed.

The problem of construction joints probably is as old as masonry construction, but with the introduction of concrete, and particularly mass concrete, it became acute. Its solution is up to the art rather than the science of engineering. It is too complex to be expressed in simple, differential equations such as Airy derived for the equilibrium of stresses in a structure. Approximate solutions for joint spacing may be obtained. However, they are of doubtful value. For example, a block between two joints may be considered as a monolith which is fully restrained from contracting at the base but progressively free to do so with increasing height. The block is assumed to be split along its median plane between the joints due to a crack, the width of which is zero at the base and equal to the full contraction due to a drop in temperature at the top. This drop in temperature must be the equivalent of all volume changes. The forces which may be applied to close the crack are governed by the allowable tensile stress. If the latter is assumed to be uniformly distributed

[63] *Proceedings*, Am. Soc. C. E., March, 1940, p. 385.

[64] Senior Asst. Chf. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[64a] Received by the Secretary August 7, 1940.

along the surfaces of the crack, then it follows that the joint spacing is directly proportional to the height of the block. According to the evaluation of the modulus of elasticity, the coefficient of thermal expansion, the drop in temperature, and the allowable tensile stress, the proportionality factor varies from about 0.6 to 1.2 and therefore would lead to wider joint spacings, especially for high dams, than are generally used.

On the other hand, if the assumption is made that the tensile stress necessary to close the crack is not uniformly distributed but is concentrated within a relatively shallow zone below the top—for example, within 5% of the height of the block—then it again follows that the joint spacing is directly proportional to the height, but the factor will vary from about 0.4 to 0.7 for values corresponding to those used before. The latter assumption is believed to approximate the physical phenomena more closely, especially in view of surface cooling, than the former. It too, however, is a crude approach to the problem as it does not account for some of the forces which are generated in the still rather mysterious setting process of concrete.

Although the attention given to the spacing of joints normal to the axis has often been wanting, it has been even more so in regard to joints parallel to the axis. Although cracks normal to the axis are not significant from a structural point of view with most types of dams, cracks parallel to the axis are very much so indeed. Such cracks have been observed in straight and curved mass concrete dams as well as in the buttresses of articulated dams.

In one case concerning a mass concrete dam in Southern California, two nearly vertical cracks parallel to the axis formed during construction, dividing a block approximately 200 ft thick, measured upstream and downstream, in three nearly equal parts. This particular block had been left low relative to the remainder of the dam to serve as a spillway during construction. Its average height was about 40 ft. A permanent and a temporary gallery crossed the block in an upstream and downstream direction. They facilitated the observation of the cracks.

Before proceeding with this lift, a mat of reinforcing steel was placed on the then existing top. It was designed to take the full shear which would develop due to full load in the plane of the cracks between the base and the mat. This was done in spite of the fact that computations indicated the closing of the cracks upon completion of the lift due to lateral expansion as expressed by Poisson's ratio. Observations in the permanent gallery substantially confirmed these computations.

Longitudinal cracks, vertical and inclined, were observed in the remaining block of the St. Francis Dam after its failure. Some of them drained water stored inside the block for several days. Whether or not all or some of them were caused by the failure, or were there before, is difficult to decide. The inclined ones near the downstream slope might well have been due to overstress at the time of the failure.

An interesting example of vertical or nearly vertical cracks occurred in the buttresses of a multiple arch dam in Arizona. These buttresses are of the hollow type as described by the late Fred A. Noetzli,[65] M. Am. Soc. C. E. The

[65] "Improved Type of Multiple-Arch Dam," by Fred A. Noetzli, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 342.

cracks were between 25 and 50 ft apart. They caused a structural weakness, particularly due to preventing the transmission of shear. To remedy this weakness five horizontal ties were installed in the buttresses of maximum height and a lesser number in the lower ones. The vertical spacing of these ties varied but averaged 40 ft. These ties consisted of a suitable number of heavy railroad rails fastened to steel yokes at the downstream face and to anchors embedded in a heavily reinforced-concrete slab immediately below the upstream or water face. All of the rails in one and the same buttress were equipped with hydraulic jacks and were pre-stressed simultaneously to from 8,000 to 12,000 lb per sq in.

This pre-stress served to cinch up the buttresses, closing the cracks and creating an initial compressive stress of about 30 lb per sq in. It was done when the reservoir was low. After completion of the pre-stressing, the rails were welded together and embedded in gunite which was bonded to the side wall. Continuous floors inside the buttresses were thereby formed.

In the design of the Big Dalton Multiple Arch Dam of the Los Angeles County Flood Control District, inclined joints were provided in the hollow buttresses. The joints follow approximately the direction of the first principal stress and are an average 50 ft apart, measured in a normal direction. Nearly vertical cracks formed in some of the buttresses. They were probably prompted by abrupt changes in the slope of the foundations, where they originated. Fortunately they are too small to be structurally significant. None of these cracks extends across the lowest joint and no cracks formed above the lowest joint. Hence, it appears as if the joints were effective and well worth the extra expense they caused.

Mr. Steele suggests the desirability of turning transverse joints from vertical to normal to the abutment walls, but points out certain practical difficulties in its accomplishment. These difficulties, particularly in regard to form work, no doubt exist and such joints, although effective, would result in slower progress than could be maintained without them. The writer believes that the shape of the abutments has much to do with cracks normal thereto. Hence, in giving due attention to the configuration of the final abutment excavation, and particularly by avoiding abrupt changes in slope as well as by avoiding convexity, most of the causes of these cracks may be eliminated. If these features cannot be provided economically, particularly in regard to a concave configuration, then the extra expense in providing normal joints might be money well spent.

Similar problems arise with concrete face slabs on embankment dams, particularly of the rock-fill type, due to relatively steep slopes and the yield under full water pressure. Diagonal pull at the abutments should be anticipated and provided for.

In contradistinction to most concrete structures, face slabs and reservoir linings of large expanse have been built without joints. In Southern California 0.5% of reinforcing steel in two principal directions has been used with much success. This prevents the formation of continuous cracks and results in the so-called turtle shell pattern of a multitude of short hair cracks.

As to radial joints in arch dams, it is believed that slots on radial planes should be used unless there are very good reasons to the contrary. With the

sections next to the slots fully cooled, and a careful job of placing concrete of low water-cement ratio in the slots, there should be need neither for keys nor for subsequent grouting.

Gravity dams, and particularly straight ones, are believed to offer a different problem. They are above all subject to sliding. This is enhanced through uplift and particularly earthquake forces. No place on earth is definitely free from earthquakes. Even if they should not originate in the immediate vicinity of the dam, they could still seriously affect it at a considerable distance. They could and probably would also affect individual blocks of such a dam. Hence, unless a straight gravity dam is designed to resist earthquake forces when under the influence of uplift, keys should be provided for the purpose of mutual support and continuity between the blocks.

The writer has not much faith in the permanence of grout in joints, nor in the permanence of metal and particularly copper seals, unless the latter are well protected and both grout and metal seals are not subjected to continuous movement. Well-designed and constructed keys or slots along vertical joints and the careful treatment of cold horizontal concrete surfaces as outlined by Mr. Blanks[45] in his six general rules are believed to be superior, particularly as to permanence. Proper treatment of surfaces between lifts was often neglected in the past. This was partly due to lack of such knowledge of the characteristics of concrete which has since been acquired through experience and research.

Generally it is believed that carefully designed joints with chamfered edges on the surface and fillets in reentrant angles, preferably with some reinforcing next to the surface, are likely to save maintenance expense and therefore justify the extra initial investment.

It is also believed that the purpose of attaining safe and permanent mass concrete structures is, aside from carefully designed and treated joints, served by using a concrete of moderate ultimate strength containing about one barrel of low-heat cement of proven quality per cubic yard and by placing it with as little water and as much care as practicable. Such a concrete with little chemical heat and excess moisture will be a homogeneous mass of uniform strength; it will satisfy the design requirements of equilibrium of stress, and the factor of safety of the structure it forms can therefore be established more definitely than with a concrete of much higher strength which lacks these essential qualities.

W. A. Perkins,[66] M. Am. Soc. C. E. (by letter).[66a]—The design of construction joints is a matter that requires far more attention from designing and construction engineers than it has received in the past, and the profession is much indebted to Mr. Steele for his timely and valuable discussion of this important subject. It can be safely said that, until recently, construction joints, like Topsy in *Uncle Tom's Cabin*, "just grew."

Merely outlining a mass of material which, by its weight or strength, will theoretically withstand the forces developed by the assumed loads does not

[45] "Treatment of Horizontal Construction Joints in Concrete to Improve Bond, Shearing Resistance and Watertightness," by R. F. Blanks, *Memoranda* to the Chf. Designing Engr., Bureau of Reclamation, March 8 and May, 1938 (not published).

[66] Senior Engr. of Dam Supervision, State Dept. of Public Works, Sacramento, Calif.

[66a] Received by the Secretary August 10, 1940.

by any means satisfy the requirements for adequate design. Many other forces enter, such as temperature, abutment and foundation yield, and differences in elastic yield in adjacent sections, which may greatly impair the strength or even wreck the structure. The best design is that which uses the minimum of material so arranged that it can readily yield to the extraneous forces such as those mentioned without losing its ability to withstand the loads which it was built to support. The proper location and design of construction joints plays a very important part in satisfying the foregoing criterion.

The writer agrees with Mr. Steele that the spacing and location of joints should be made to conform as closely as possible with major breaks in the profile of the site after the excavation has been completed. Examination of dams where this rule has not been followed will disclose, in many if not most cases, a diagonal crack extending into the body of the dam where there is an abrupt change in profile. Several factors combine to cause this condition, among them being the differential elastic compression of the concrete and the adjacent rock of the abutment as the load of the upper portion of the dam is imposed and local stresses occur due to difference in elastic yield when water load comes upon the dam, whether arch or gravity in type.

Seldom will the location of outlet works and other features be interfered with by the insertion of an irregularly spaced transverse joint to satisfy an irregular condition in the profile of the dam, as this will always occur either at the foot of an abutment or well up above the bottom. The slight additional expense for form work would be a very small percentage of the cost of the dam and is scarcely to be considered.

Experience would indicate that it is advisable to begin all radial contraction joints at the foundation. Where some of the joints are placed only in the upper portion of the dam, uncontrolled cracks are likely to develop and extend downward, usually in a diagonal direction from the lower end of the constructed joint, a condition noted by the writer in several dams.

Architectural considerations need not be a bar to selecting joint spacing that will conform to conditions at the site as attested to by the fine appearance of the downstream face of O'Shaughnessy Dam in which there was variation at the abutments from the regular joint spacing.

Discussion of the joints in O'Shaughnessy Dam has been invited by Mr. Steele, and as the writer is somewhat familiar with that structure, a few notes thereon are inserted here. A brief description of the dam follows. Elevations are: Lowest foundation, 3,382; stream bed, 3,500; crest of first stage of construction, 3,726; and crest of completed dam, 3,812. The dam is curved in plan to a radius of 700 ft at the upstream face, and is full gravity type in cross section. The crest length of the original dam was 605 ft and of the completed dam 900 ft.

Joints in the original dam were placed radially with a spacing at the upstream face of 97 ft. When the dam was raised to its final height, these contraction joints were continued to the downstream face, and midway between them an additional joint was placed. At the ends, especially the right, configuration of the abutments after excavation required a different spacing. As

stated previously, however, this does not mar the excellent appearance of the downstream face.

Between the old and the new concrete a chamber of an average width of about 5 ft was formed extending from the base of the new work to the crest of the old dam. The inclined weight of the new concrete was carried to the old on ribs 2 ft wide adjacent to the contraction joints and poured monolithic with the new work. Horizontal drains were placed at the top of each step of the old dam, and all surfaces of the old concrete were thoroughly roughened. Likewise, the face of the new concrete adjoining the slot was so formed that, when the slot was filled with concrete, the maximum resistance to movement due to shear, both of the cantilever and of the arch, was provided.

After the new concrete had been cooled to about mean annual temperature, 50° ±, the slots were filled and cooled, and the remaining concrete from El. 3,726 to 3,812 was poured for the full width of the dam, no longitudinal joint being provided above the top of the slot at the crest of the old dam.

Settlement of the new concrete between El. 3,500 and El. 3,726, due to cooling and elastic yield as additional concrete was added, could readily be observed by noting the development of cracks in the supporting ribs, these cracks in practically all cases originating at the outer corners of the old steps.

At the upstream edge of all contraction joints, and also along the crest of the old dam, U-shaped copper water stops were provided. Likewise, grout stops were provided whenever necessary in all the new concrete, including horizontal stops at 50-ft vertical intervals. However, to provide for grouting all joints, both old and new, it was necessary to calk all joints in the old dam, except where water stops were used, and also in many places where the old concrete adjoined the new. As there were no horizontal stops in the old dam, it was necessary to exercise great care in bringing these joints under hydrostatic pressure before inserting grout to avoid side deflections of the blocks, thus allowing the calking in the joints that opened to loosen. Aside from a little trouble because of this loosened calking, no difficulties were encountered in the grouting operations.

Numerous inspection wells and galleries are provided in this dam, including inclined galleries along all the contraction joints of the old dam at the contact between the new and old concrete, from El. 3,500 to El. 3,730.

An inspection of the dam was made by the writer soon after the completion of the new work with water within 4 ft of expected maximum storage level. The total observable leakage at that time would not exceed 50 or 60 gal per min, and the major portion of this came from step drains near the right end and was believed to enter through a crevice in the rock of the abutment which was not entirely sealed by the grouting operations. Another inspection made recently, with water surface about 30 ft lower than at the first inspection, revealed a leakage about half as great.

Observation of the supporting ribs adjacent to the inspection galleries and readings of strain meters so far indicate no differential movement between the new and old concrete or any other action contrary to expected behavior. It is fortunate that in this dam the "sun face" is on the downstream side where the greater movement due to the wider range of temperature change is provided

for by the additional joints placed in the new work. Since completion of the work, in 1938, no cracks have been discovered so far as the writer is aware.

The writer fully agrees with Mr. Steele in his statement concerning longitudinal joints. This is a matter which has not received sufficient consideration in the past, but with the increase in the size of dams it should be given more attention. Cracks develop sometimes in the most unexpected places, and a longitudinal crack in a large gravity dam may be so located as to seriously impair its stability. Furthermore, the probability of discovering an internal longitudinal crack is much less than observing the development of a transverse crack. This is a matter which merits the serious joint consideration of designing and construction engineers.

Mr. Steele questions the advisability of the use of keys in joints. It is true that this may be a heritage from the past, just as the early paved roads inherited their high crowns from the old dirt roads. However, this is not sufficient justification for scrapping the keys entirely. In the case of arch dams engineers need to have assurance that the sections of the dam will lock together securely to produce the proper arch action before cantilever action due to rapidly rising water against the higher sections has produced dangerous vertical beam stresses.

The use of keys and their type is a matter of design fully as much as any other part of the structure. In considering this feature, certain questions are pertinent. Will the joint be weakened by the use of keys? Does the proposed type best serve the purpose for which it is intended? If keys are omitted, how will the structure behave when placed under load? In arch dams having heavy overhang at the crown, a proper system of keys is necessary in order to give stability to the structure through arch action without the development of undue cantilever stresses until such time as the dam can be properly grouted. The use of slots in this type of dam is not practicable.

Use of slots instead of contraction joints for the closure of a dam does not necessarily guarantee that transverse cracks will not develop at some subsequent time. In this respect, the use of joints with adequate water stops at the upstream face better permits the movements to take place at controlled points, eliminating the formation of uncontrolled cracks.

As the weakest link determines the strength of an entire chain, so do joints between successive pours of concrete determine the strength of a dam or other structure. The wide divergence of opinion among engineers as to the best method of cleaning cold joints indicates the interest that constructors are taking in this feature of the work.

The cleanup really begins when a pour is being completed. It is highly important that the moisture content of concrete be sufficiently low to prevent the accumulation of surface water and flotation of the lighter particles of the cement. Vibration should be continued a sufficient length of time to consolidate the mass, and surface working should be limited to that necessary to eliminate gravel pockets and loose pieces of aggregate. This should be done by forcing the pieces of aggregate into the mass by booting or with tampers rather than bringing fine material to the surface by puddling. After pouring,

the surface of the concrete should be protected from all disturbances, after initial set has begun, until it is sufficiently hard not to be injured by traffic.

Specifications should be so written that the construction engineer will have sufficient latitude to select that method of cleanup which will give the best results, and he should not leave this important item of the work to the care of an inexperienced inspector. With proper manipulation, performed at the right time, good surfaces can be secured by any of the various means that have been used. On the other hand, the best method yet devised will not be satisfactory if slipshod methods of operation are used.

In the opinion of the writer, the mortar that is spread on a cold joint ahead of a concrete pour should be of practically the same mix as the mortar of the concrete that is being used. It would be interesting to do experimental work with gunite on a cold joint to study this type of treatment. Instead of using a layer of mortar it is suggested that a thin layer of rich gunite, using fine sand, be spread on the hard joint surface just ahead of the advancing face of the new concrete. The cost should be no greater than the use of mortar, and the results might be more satisfactory.

In gravity dams, where sliding is important, the factor of safety is increased by sloping the joint surface slightly upward toward the downstream face. This increase amounts approximately to twice the tangent of the angle of the slope.

There is still room for much improvement in the design of satisfactory water stops at construction joints. There is serious objection to the type commonly used in the United States—namely, a copper sheet bent in a U-shape with two long wings buried in the concrete. If a leak develops at a brazed joint or by a piece of sharp aggregate piercing the plate during movements of the dam, repair is impossible, and the value of the entire water stop is lost. Also, unless the stop is buried deeply, there is danger of the concrete outside the embedded wing breaking off. In the case of a thin arch dam, although it is designed on the assumption of the arch loads being carried by the full thickness of the concrete, yet at each vertical construction joint a large percentage of this thickness is lost because of the necessary depth at which the water stop and, on the downstream face, the grout stop are buried.

A bent copper plate with an exterior protective plate held in place across the joint by liner bars and anchor bolts is one substitute that could be used. A design embodying this idea was used at the Bissorte Dam, shown in Fig. 13 of Mr. Steele's paper. It is believed that a design could be developed, however, that would be somewhat less expensive than that illustrated in Fig. 13.

Construction joints are an important feature of all concrete structures, especially dams, and merit the most careful consideration of designers and specification writers. Their work, however, is completely nullified without the intelligent "heads-up" cooperation of a competent and alert construction organization.

L. J. MENSCH,[67] M. AM. SOC. C. E. (by letter).[67a]—High dams exert enormous pressures and shears on the foundations and abutments. Nearly every

[67] Civ. Engr. and Constructor, Chicago, Ill.

[67a] Received by the Secretary July 5, 1940.

contributor to this Symposium has laid particular stress on the importance of the thorough exploration of rock conditions surrounding the dam so that proper grouting procedure may be planned in order to make the foundations as nearly as possible "bedrock."

This perfect agreement between experts leads one to the conclusion that serious troubles must have been encountered with some high dams after they had been built. Messrs. Houk and Keener refer to cracks, seams, fissures, joints, and bedding planes in all masses of rocks, in view of which the tests on cubes of sample rock which are generally very carefully faced on all six sides will hardly give a proper answer to the question of the actual strength of rock, in place.

Ultimate strengths of test cubes vary from 3,000 to 45,000 lb per sq in. in compression when tested dry. Many rocks absorb as much as 10% of water, and test cubes, when tested wet or with oiled surfaces, will show strengths of only a fraction of those tested dry. Shearing strengths of rocks are very low —as low as 150 lb per sq in. in some granites and probably not more than 1,000 lb per sq in. in the best granite ever tested.

Values of safe shearing stresses of 300 to 700 lb per sq in. as mentioned by Messrs. Houk and Keener are astoundingly high, and when considering the dip of the bedding planes of some granites or similar rocks, the only safe assumption is to consider the base as so much sand, when the shear-friction factor of 0.65 will appear rather high than low.

A high dam like Boulder Dam, if it acts like a gravity dam, exerts a horizontal shear of 8,000 tons for each linear foot of the longitudinal axis of the dam. How is this great force transmitted to the shattered and partly grouted masses of rock beneath, downstream, and sidewise? One can imagine secondary horizontal or inclined arches acting in the rock which are affected by the horizontal shearing forces and by the inclined compressions from the downstream side of the dam. The rock may be weaker than the concrete, especially downstream and to the side of the canyon where no grouting is done, and the secondary arches will be very much wider than the base of the dam. If such arches are forming, then one can see the necessity for extending the grouting operations far beyond the base of the dam.

There is no doubt that any dam, whether a gravity or arched structure, with proper abutments will act like a curved plate; but this plate has cracks and joints in three directions and the only safe way is to judge the structure as if built of jointed brickwork. Tests on reinforced brick beams have shown[68] that such jointed work fails when the shear reaches a value of about 40 lb per sq in. The 8,000 tons acting in Boulder Dam, provided it were a gravity structure, on a surface 1 ft wide and 650 ft long (the width of the base), mean that the shear would be 171 lb per sq in., a value probably four times as high as a jointed structure could sustain. Hence, it is evident that Boulder Dam acts as a curved plate and not as a gravity structure, notwithstanding that the base is nine tenths of the height.

However, assuming that this curved plate were an entire monolith, there are some very serious movements of which Messrs. Houk and Keener did not

[68] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 755.

make any mention. Assume near the bottom of the dam a prism or prismoid cut out of the dam, 1 ft square and 650 ft long, the 1-ft end to be at the upstream face of the dam, and the slightly smaller end at the downstream face. The water pressure acting at the base of the dam is 45,000 lb per sq ft, which will act on the upstream face of the prism and this pressure will decrease to zero at the downstream end. Lamé's theory of thick pipes acted upon by outside pressures will afford an idea as to how the compressions on the cross sections of this filament will diminish with the distance from the upstream face. The compression will decrease more slowly than according to a straight-line variation from a maximum at the water side to, say, zero at the air side. Supposing that the average compression is only $\frac{1}{2} \times 45{,}000 = 22{,}500$ lb per sq ft, the shortening of this prismoid will be given approximately by $\frac{22{,}500}{144} \times \frac{650 \times 12}{2 \times 10^6}$ $= 0.61$ in.; this is the relative movement between the two faces of the dam.

Unless the underlying rock has the same movement between the two faces of the dam, a shear crack must open either in the concrete above the rock or in the rock beneath the concrete. The downstream face of the dam has its own deflection, however, whether it acts as a horizontal arch or a horizontal beam, which will still further induce relative movements between dam and foundations. Conditions are certainly serious at the contact of dam and foundations in a dam of the Boulder type. In a dam only 360 ft high, the movement will be only one fourth as large.

Tangential shear, twisting moments, and beam actions are effects which are very little understood even by high-class stress analysts. These actions have been considered by A. E. H. Love[69] for a thin elastic plate. A practical analysis by H. Leitz,[70] based on Professor Love's theory on the design of rectangular plates supported on four sides, led to erroneous results which differ from test results by several hundred per cent. Professor Love shows how the twisting moment may disappear at any point of the curved plate by finding the section in which the principal stresses are acting. How this can be done in a simple way has been demonstrated by Prof. Arturo Danusso[71] for both rectangular and triangular plates. Professor Danusso's formulas agree exceedingly well with actual tests to destruction.

Where the canyon is of a rectangular shape, the system of horizontal arches and vertical cantilevers is the best approach for a proper stress analysis; but where the canyon approaches a triangular shape, the arches in the lower part of the dam should be assumed inclined, on the lines indicated by the writer,[72] and the cantilevers should be assumed normal to the foundations and abutments.

As far as the writer can see it will take many years yet before the stress problem of the high dam will be solved to the satisfaction of conservative engineers, and agencies responsible for the design of large dams should give serious consideration to types of dams other than the solid gravity or solid arched dam, especially where the foundation conditions may not be ideal.

[69] "The Mathematical Theory of Elasticity," by A. E. H. Love, Cambridge, 1906.

[70] "Forscherarbeiten auf dem Gebiete des Eisenbetons," William Ernst & Sohn, Berlin, Heft 23, 1914.

[71] *Il Cemento* (1911) (first 10 issues).

[72] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1262.

The paper on concrete control in this Symposium is a fine exposition of the present stage of concrete practice in dam construction.

The paper on construction joints by Mr. Steele is highly interesting and shows the perplexing nature of joints and cracks. Mention is made of cracks forming during concreting and very soon afterward. They are the effect of volume changes due to chemical actions, moisture, and temperature variations and very often, on hot days, to the sun rays beating down on the fresh concrete or on the forms. Much less cracking occurs in winter than in summer work.

Joints and cracks cannot be prevented in dams, and the designer must take this fact into consideration. As stated by Mr. Steele, keyed joints in concrete dams and retaining walls (and tanks) are a hang-over from the mortise and tenon used in timber construction, and the fourth paragraph under "Plain Versus Keyed Joints" deserves to have been printed in bold face type. This detail is one of the worst features in common design practice. Such keys are expensive to build; the depressions form sinkholes for the accumulation of dirt, wood shavings, etc.; they are difficult or impossible to clean out; and cracks are forming either from shrinkage or the removal of forms. The tests by M. Considère[73] on closed joints with steel dowels have demonstrated that such joints fail at a unit stress in the dowels of from 30,000 to 40,000 lb per sq in. Dowels form an inexpensive means for making joint connections, but no such mention has been made in any paper of the Symposium; therefore, one must conclude that designers have not yet used dowels to any extent in joints of dams.

Water stops of metal are another perplexing problem. Such stops have not proved successful in road construction and have been abandoned by most highway departments after a trial for several years. Neither have they been successful in high dams. No tight contact between the flanges of the seal and the concrete can be counted upon, even where vibration is used. Tiny air bulbs seem to stick to the metal and prevent contact, or in case of vibration, air will accumulate at the flanges. Movements of the adjoining concrete surfaces have caused cracks in the crimped portion of the seal, thereby making it worthless as a water stop. The flanges cause cracking of the surrounding concrete, and the wider the flanges the more serious the cracking will be.

Asphaltic joint fillers have not proved a success in most joints; where there are large movements the filler has been squeezed out to such an extent that the bare concrete surfaces have been found in contact. Boards or planks of wood with protective treatment make a cheaper and better filler; under a pressure of 1,000 lb per sq in. across the grain they will compress to one half their thickness without extrusion.

Lewis H. Tuthill,[74] Esq. (by letter).[74a]—In his comprehensive paper and references Mr. Tyler has given an excellent résumé of the fundamentals of good concrete making for masonry dams. There are two aspects of this subject, however, which may well receive brief amplification in discussion.

[73] "Expériences, rapports et propositions instructions ministérielles relatives à l'emploi du béton armé," Commission du ciment armé, Paris, Dunod, 1907.

[74] U. S. Bureau of Reclamation, Denver, Colo.

[74a] Received by the Secretary July 12, 1940.

The first point is emphasis on the importance of uniformity of aggregate as the very foundation of concrete control. It is true that concrete in place may be greatly benefited by use of vibrators, special cements, cooling systems and absorptive form lining regardless of the uniformity of the aggregate as batched; but these highly important elements do not constitute the control which is responsible for the efficiency and the uniformity of fresh concrete as a manufactured product for use as a construction material. The most efficient concrete is that having the minimum cement and water contents compatible with adequate workability, sufficient strength, durability, and other essential properties. Unless aggregate is uniform in grading, cleanness, and in silt and undersize content, an appreciable extra margin of cement (and possibly of water also) must be provided to insure quality and workability under all conditions of aggregate variation. Uniformity needs no definition, and its importance as a property of concrete should be obvious.

The now recognized practice of using five or six divisions of aggregate for mass concrete is a long step toward aggregate uniformity as batched when compared with the fairly recent use of two or three sizes with the one division of coarse aggregate regarded by some as a needless refinement. Screening equipment has become more efficient, and current specifications are usually more particular about the efficiency of screening and percentages of undersize. On larger work, however, the necessity for an ample standby supply, and for more frequent rehandling of the material between aggregate plant and batchers, often results in a variable undersize problem which, at times, may largely nullify most of the efforts made to produce uniform concrete. Variable quantities of fine gravel from among each of the sizes are principally responsible for this difficulty.

The percentage of pea gravel (No. 8 to $\frac{3}{8}$ in.) in a concrete mix is the most critical of the several aggregate sizes, particularly in mass concrete, because an efficient but amply workable mass mix contains less mortar than corresponding mixes of smaller aggregate. With lower mortar content there is consequently a smaller surplus available to overcome changes in the aggregate which demand more mortar if workability is to be sustained. The percentage of pea gravel is critical because more mortar is required to maintain workability when a unit of pea gravel is added than for a similar unit of any other size of coarse aggregate. On account of breakage and imperfect screening each bin of a coarse aggregate size is a source of pea gravel that varies in percentage depending on how the undersize aggregate may accumulate and vary in its flow to the gates (see Fig. 24). Material coarser than No. 8 in the sand is another source, and it may vary in percentage with changes in grading of the source material. Since variations in the quantity of pea gravel entering the batch may vary as much as 100% from that desired, due to certain unfavorable combinations of situations in the various aggregate bins, it is clear that if uniformity is to be obtained means must be established for keeping the smaller fractions of undersize at a minimum and flowing uniformly into the batchers batch after batch. (Undersize in larger aggregate that is but little smaller than those sizes is usually not significant since equivalent proportioning changes in the mix seldom produce a noticeable difference.)

Excessive percentages of pea gravel sizes in sand due to the grading of the deposit can be removed by proper screening processes, and that remaining in wet sand will give little or no trouble due to segregation in the bins. In dry sand, however, any appreciable percentage of material larger than No. 8 will segregate in handling and in the batcher bins, and will give very serious trouble. The most effective correction in this case is to use a $\frac{1}{8}$-in. screen for sand and to put the pea gravel (screened through a $\frac{1}{2}$-in. or $\frac{5}{8}$-in. screen) in a separate bin. In fact, much better control is afforded with either wet or dry material by keeping pea gravel of these sizes separate where aggregate in the mix is graded to a maximum size larger than 1 in.

The solution of the difficulty from undersize (after due provision has been made to keep breakage minimum for the sake of preserving the usually deficient

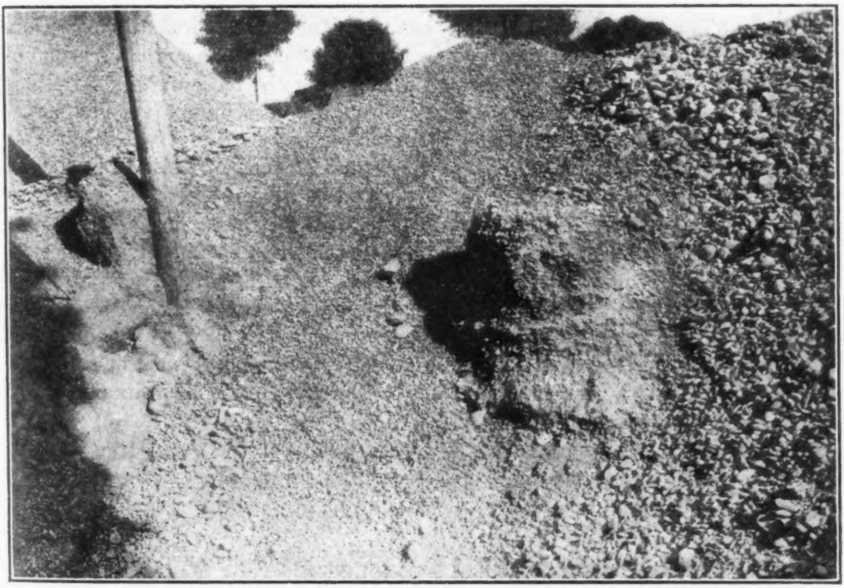

FIG. 24.—CONCENTRATION OF UNDERSIZE AGGREGATE AT POINT OF DISCHARGE INTO STOCK PILE

supply of large sizes) is finish-screening at the batcher bins. When this is done all previous screening can be very rough, and the usual overlapping and segregation in economical conical stock piles over conveyer tunnels can be ignored. By adjustments in simultaneous feeding of all sizes on to the belt from the stock piles supplying the finish screen, the desired balance of finished materials in the batcher bins can be maintained. Since any undersize or oversize will not correspond in grading to the specified next smaller and larger sizes, no one size should be screened singly over the batcher bins unless dependable and effective means are provided to insure the uniform blending of undersize and oversize with the next smaller and larger sizes.

By following the practice of finish-screening over the batcher bins during construction of the Colorado River aqueduct, the difficulties commonly result-

ing from stock-piling were largely avoided to the benefit and satisfaction of both construction and engineering interests, and the practicability of such procedure was demonstrated. Operations requiring materials for as much as 100 cu yd of concrete per hour were efficiently supplied in this manner and with surprisingly few delays, although only one screening unit was used over each plant.

It is recognized that there must be no delay in the supply of aggregate after concreting starts, especially on a very large dam. Adequate protection from delay due to finish-screening at the batching plant may be obtained (1) by maintaining the bins over the batchers as full as possible at all times; (2) by somewhat increasing the ordinary capacity of the bins over the batchers; and (3) by dividing the installations for finish-screening into two, three, or more parallel, independently operating sets of equipment having a combined output well in excess of average requirements. Delays in aggregate excavation, processing, and transportation may be absorbed by use of the common, economical conical stock piles over a conveyer tunnel near the batching plant. Since handling of finished material regularly through this ordinary type of large-volume storage develops objectionable variable segregation of the character shown in the illustrations, and standby ground storage of finished aggregate (unused except in an emergency or at the end of the job) involves expensive handling operations, the provisions suggested for avoiding serious delay in connection with the proposed finish-screening are relatively inexpensive in view of the superior uniformity of aggregate obtained. It is recognized that in many cases it may not be economically feasible to change over established job setups in order to do the finish-screening at the batching plant. For best economy and results the aggregate processing, handling, and batching plant must be arranged at the outset for this plan of operation. Perhaps it should have its beginning as a specification requirement.

For structural economy in the batching plant for the very large jobs, it may be necessary to locate the finish-screening equipment adjacent to, rather than above, the batching plant. Conveyers or elevators for each aggregate size should then be provided.

Except where materials are dry, the sand should not be finish-screened with the coarse aggregate but should be separated from it and washed, and processed if necessary, at the aggregate plant. This leaves three alternates for disposition of undersize in the first size of gravel larger than sand: (1) It may be wasted; (2) it may be arranged to blend uniformly with the sand when the latter is being run into the sand bin (provided it does not impair the grading of the sand); or (3) the range of limiting dimensions of the fine gravel may be reduced (to $\frac{1}{2}$ in. or $\frac{5}{8}$ in. maximum), and as a result its percentage in the mix may be limited, such that variations in its percentage of undersize will be unimportant, making a finish-screening on its minimum size comparatively unnecessary.

When finish-screening is performed at the batching plant, screening at the aggregate plant may be considerably simplified. Aside from separating the sand (if it is a wet plant), scalping oversize, and wasting excess in certain sizes (often a portion of the sand and pea gravel), little screening of coarse aggregate need be done except perhaps roughly into two or three sizes in order to expedite

later adjustment of the feed from the stock piles to the finish-screens so that material will maintain a balance in the batcher bins. Whether roughly separated or not, if the coarse aggregate may be drawn through a number of adjustable gates under the storage piles, the operator will usually be able to maintain a good balance among the aggregate sizes.

The other point which might be mentioned is the remaining (though narrowing) lag between what is known and should be done about concrete control, and what actually is done about it on the job. Under the heading, "Conclusion," Mr. Tyler says: "Efforts toward * * * control * * * are becoming more successful * * *." It is not clear whether he means that this is because control methods in themselves are improving and are better applied, or that control methods as such are being adopted more readily by management. It does not greatly matter which, since either means a better net result, but why should there be, or have been any serious question on the part of management as to the desirability of practicable measures of control? Too often progress has been impeded and opportunities for obtaining superior results have been lost by an attitude of management toward control indicated by such remarks as "If we let these cranks have their way they will ruin the job." If this were only the attitude of the occasional superintendent it would be no cause for comment, but to a various degree it is yet to be found among engineers, and they, of all people, should be aware by this time of the true meaning and value of concrete control, and should lend their considerable influence toward an invariable application of its simple principles. When management recognizes that concrete control is merely the application of all practicable means of getting the most out of the concrete construction dollar, the costly lag between what is known and what we do will be appreciably shortened. Mr. Tyler may then say, "Efforts toward control will be successful."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

# OXYGEN CUTTING (FLAME CUTTING) OF STRUCTURAL STEEL

## PROGRESS REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION

### Discussion

BY H. H. MOSS, ESQ.

H. H. MOSS,[3] ESQ. (by letter).[3a]—The Committee has studied the processes of oxygen cutting of structural steel and has stated the facts of current experiences correctly and concisely and its conclusions are indeed to the point and in line with the experiences of the laboratory with which the writer is connected. The writer is confident that oxygen cutting will have no adverse effect on the fatigue properties of structural grades because, when normally applied, oxygen cutting is grain refining. If followed by proper tempering, such as imparted by the more recent localized heat treating processes, very excellent properties can be imparted to the refined zone, promising good fatigue resisting characteristics.

Several recent developments in oxy-acetylene cutting promise still greater facility to the steel fabricator, such as flame gouging, precision cutting, and compound and profile cutting in plate edge preparation. These have already won a definite place in steel construction work at a number of points.

NOTE.—This Progress Report of the Committee of the Structural Division was published in April, 1940, *Proceedings*.

[3] Development Dept., The Linde Air Products Co., Newark, N. J.

[3a] Received by the Secretary May 29, 1940.